VOL. LI. MARCH, 1925. No. 3.

REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

This Society is not responsible for any statement made or opinion expressed in its publications.

CONTENTS

For Index to all Papers, the discussion of which is current in *Proceedings*, see the second page of the cover.

SHERMAN ISLAND DAM AND POWER-HOUSE

By H. deB. Parsons*, M. Am. Soc. C. E.

To Be Presented March 4, 1925.

Synopsis

This paper describes the design and construction of the Sherman Island Dam and Power-House on the Hudson River, about 8½ miles west of Glens Falls, N. Y. The development, built in 1921-23, is an hydro-electric plant with an ultimate capacity of 50 000 h.p. under a normal head of 66 ft. There are four 10 000-h.p. generating units now in operation; when additional regulation of the river is provided, the fifth unit will be installed.

The dam is 80 ft. 9 in. high and has a total length including the spillway and head-works of 2 069 ft. The lower half of the up-stream face is on a slope of 5 on 12, to take advantage of the weight of water on the dam in increasing the resistance to sliding.

Geologically, the site was affected by at least two heavy glacial actions, and a deposit of sand and boulders extends to great depth over a wide area. As a result, the development had many interesting features, both in design and construction. The main dam, of the multiple-arch type, was founded on sand and boulders with interlocking steel piles to form a cut-off. Pipes were placed through the base and the uplift pressure was determined. The spillway dam, horse-shoe shaped in plan, has a gravity section and is founded on rock. The head-race canal, 3 800 ft. long, was in fine sand and lined with concrete. The power-house was built on very fine sand and considerable difficulty was experienced in excavating its foundation.

As there are few descriptions of dams and power-houses of this size founded on pervious material, a record of its salient features is given so that others may benefit. A further interesting feature is the measurement of the uplift pressure.

General Description

The Sherman Island Development, built for the International Paper Company between 1921 and 1923, is a 50 000-h.p., hydro-electric plant, on the Hudson River about 8½ miles up stream from Glens Falls, N. Y., and about 4 miles down stream from Spier Falls, N. Y.

Note.—This paper is issued before the date set for presentation and discussion. Correspondence is invited and may be sent by mail to the Secretary. Discussion on the paper will be closed in May, 1925, and, when finally closed, the paper, with discussion in full, will be published in *Transactions*.

* Cons. Engr., New York, N. Y.

The profile, Fig. 1, shows the location of the Sherman Island plant with reference to other power developments on the river between Palmer Falls and Fort Edward, N. Y. The Paper Company owns the multiple-arch dam at Palmer Falls and the crib dam at Fort Edward, and is part owner of the multiple-arch dam at Glens Falls and of the recently built power plant at Feeder Dam. The Spier Falls Dam, owned by the Adirondack Power and Light Corporation, and the Sherman Island Dam were designed by the writer, who also was Consulting Engineer for the designs of the Palmer Falls and Glens Falls Dams. The normal working heads are 84 ft. at Palmer Falls, 80 ft. at Spier Falls, 66 ft. at Sherman Island, 15 ft. 6 in. at Feeder Dam, and 46 ft. at Glens Falls.

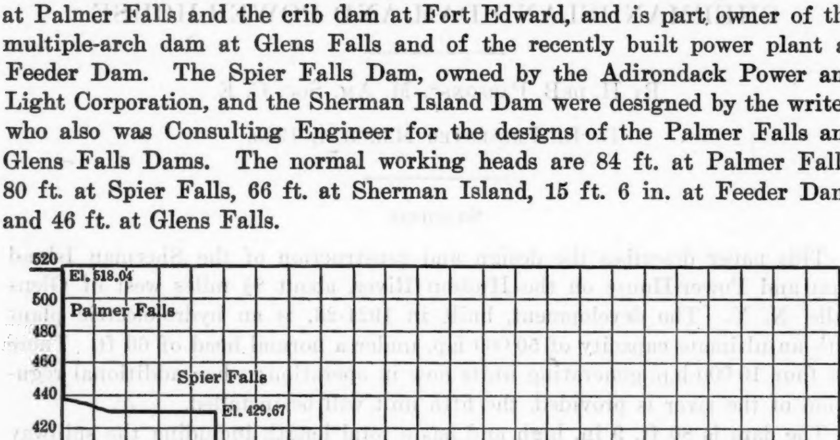


Fig. 1.

The general plan for the development of power at Sherman Island was prepared by A. H. White, M. Am. Soc. C. E., Chief Engineer of the International Paper Company. The writer was asked in November, 1920, to report on the main and spillway dams, and he suggested cross-sectional designs for these structures, which were accepted by the Company. Mr. White then prepared designs for the remainder of the development (Fig. 2), worked out the details, and supervised the construction.

The original plan called for a power-house at the north end of the main dam. To secure the additional head due to the rapids below the dam, a tail-race was proposed through the channel between the north bank of the river

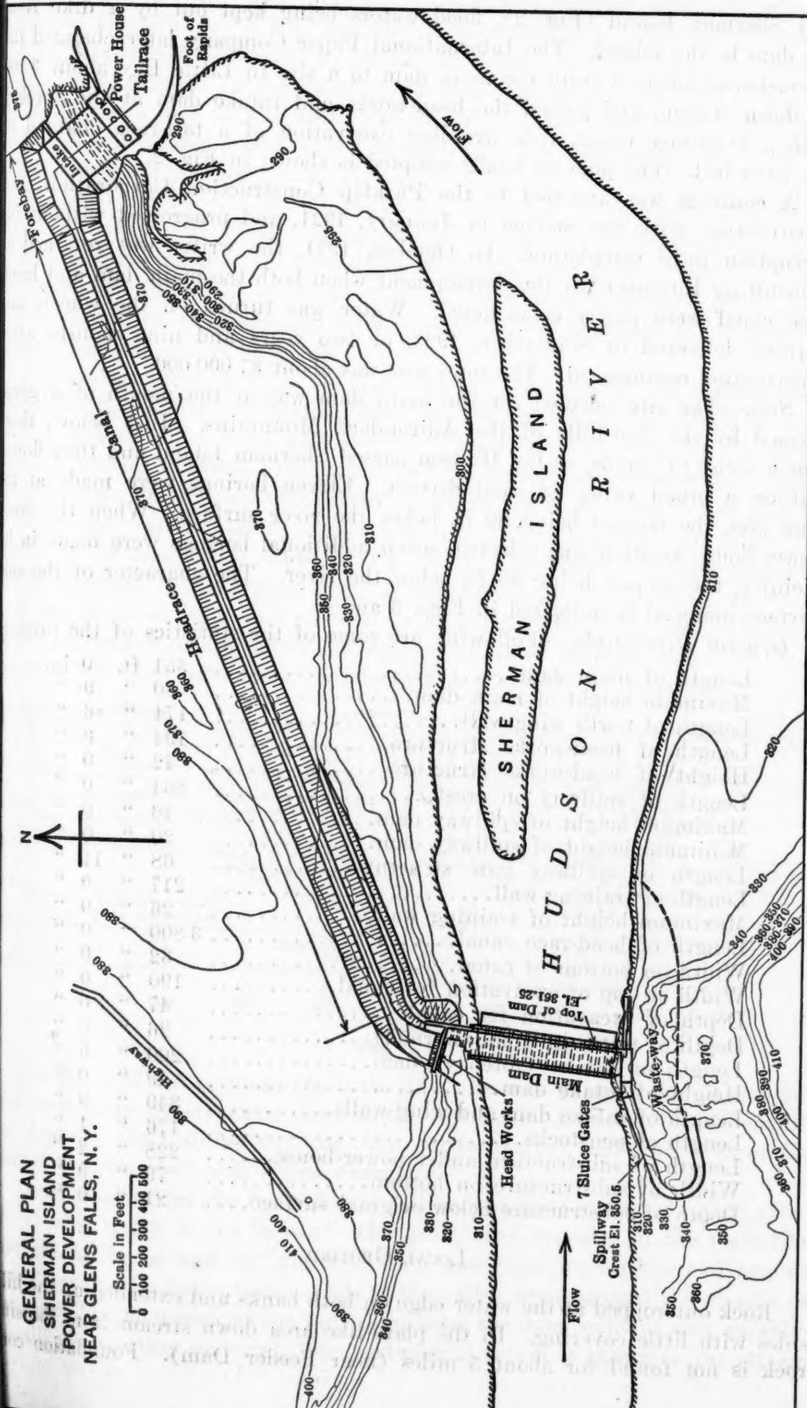

GENERAL PLAN
SHERMAN ISLAND
POWER DEVELOPMENT
NEAR GLENS FALLS, N. Y.
Scale in Feet
0 100 200 300 400 500
N
Power House
Tailrace
Foot of Rapids
Intake
Forebay
Canal
Headrace
Head Works
7 Sluice Gates
Main Dam
Top of Dam
El. 361.25
Spillway
Crest El. 350.5
Waste-way
Flow
Flow
SHERMAN ISLAND
HUDSON RIVER
Highway

FIG. 2.

and Sherman Island (Fig. 2), flood waters being kept out by a dike from the dam to the Island. The International Paper Company later changed the power-house location from the main dam to a site in Little Bay about 3 800 ft. down stream, and joined the head-works and intake dam at the forebay with a head-race canal, thus avoiding excavation of a tail-race channel in the river bed. The plan as finally adopted is shown in Fig. 2.

A contract was awarded to the Parklap Construction Corporation, and construction work was started in January, 1921, and progressed without interruption until completion. In October, 1921, the writer was retained as Consulting Engineer for the development when both the main dam and head-race canal were partly constructed. Water was turned on the wheels and current delivered in September, 1923, or two years and nine months after construction commenced. The total cost was about $7 000 000.

Site.—The site selected for the main dam was at the mouth of a gorge formed by the foot-hills of the Adirondack Mountains. Just below, there was a series of rapids, as the Hudson passed Sherman Island and then flowed out on a broad valley of sand deposit. Eleven borings were made at the dam site, the deepest being 80 ft. below the river surface. When the lower power-house location was selected, seven additional borings were made in its vicinity, the deepest being 90 ft. below the river. The character of the subsurface material is indicated in Figs. 3 and 4.

General Dimensions.—Following are some of the statistics of the project:

Length of main dam	551 ft.	0 in.
Maximum height of main dam	80 "	9 "
Length of north wing-wall	174 "	6 "
Length of head-works structure	194 "	0 "
Height of head-works structure	42 "	0 "
Length of spillway on crest	864 "	0 "
Maximum height of spillway dam	46 "	0 "
Minimum height of spillway dam	20 "	0 "
Length of spillway gate structure	68 "	11 "
Length of training wall	217 "	0 "
Maximum height of training wall	26 "	0 "
Length of head-race canal	3 800 "	0 "
Width on bottom of canal	32 "	0 "
Width on top of excavation for canal	190 "	0 "
Depth of excavation for canal	47 "	0 "
Depth of water in canal (normal)	26 "	0 "
Length of intake (forebay) dam	200 "	6 "
Height of intake dam	49 "	0 "
Length of intake dam and wing-walls	349 "	9 "
Length of penstocks	176 "	1 "
Length of substructure under power-house	228 "	2 "
Width of substructure on bottom	79 "	0 "
Depth of substructure below original surface	21 "	0 "

Local Geology

Rock outcropped at the water edge on both banks and extended up the hill-sides with little covering. In the plain-like area down stream from the site, rock is not found for about 5 miles (near Feeder Dam). Foundation con-

ditions change at the dam site, there being two entirely different sets of geological formations. A fault crossed the development between the main dam and the upper end of Sherman Island, on a line practically parallel to the dam. Rock, exposed in the canal excavation just behind the head-works on the north bank and in the waste-way channel on the south bank, exhibited fractured and twisted strata caused by this slip. The part broken off is buried at unknown depth under a sand deposit.

The geologic floor is the crystalline rock of the Adirondack Region, of complex origin and exhibiting great variability; it is commonly called Adirondack gneiss, and its interstices are filled with fused quartz. It has been worn and eroded by at least two glacial periods of prehistoric times, and plainly shows ice markings and scratches; it is hard, durable, and altogether suitable for foundation purposes.

The geologic rock floor in the river bed has been eroded and covered with glacial drift, solidly compacted, its upper layer containing boulders and cobbles embedded in sand and gravel. These boulders vary in size from ordinary cobbles to about 1-yd. stones, and are smooth and water-worn. Many are of foreign origin, probably carried down by the slow movement of the ice from remote regions; their deposit evidently marks a ground moraine. A section at the dam site is shown in Fig. 3. The pavement of larger boulders, chiefly confined to the upper level, is approximately 15 to 20 ft. in thickness, and forms an important element in supporting the main dam. The compacted material beneath, offers obstruction to water passage under the dam, and this "till" has been found satisfactory for foundation purposes.

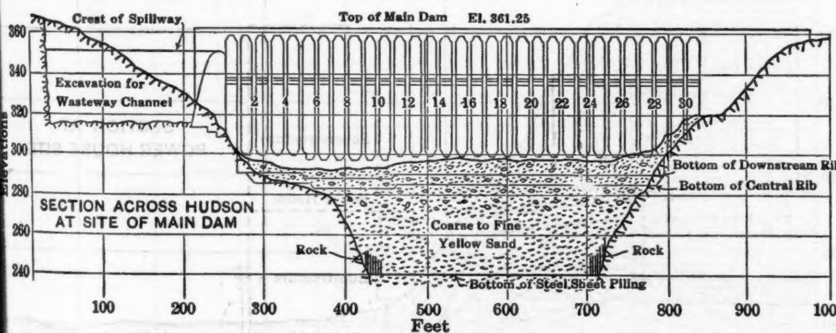


FIG. 3.

The depth of the gorge at the main dam site was so great that it was not feasible to carry the structure down to the rock floor. However, both the spillway dam and the training-wall for their entire lengths, and both ends of the main dam rest on bed-rock of firm character.

Down stream from the fault the crystalline rock floor is lost. It must exist, but at what depth is not known. The great sand deposits in the plain-like area are the "scour and fill" material from water and ice workings of bygone ages. Apparently, the outwash sands of the valley come against the margin of the crystalline floor rock at the fault line, near the dam, without any underlying till or transition material. The post-glacial action is shown in the stream

erosion of the finer materials, leaving the boulders as a pavement along the river bed. This boulder pavement, placed by Nature, has exhibited resistant qualities against dislodgment and scouring by floods of centuries. At Sherman Island, the obstruction was sufficient to cause the river to cut channels on either side, and leave a high area which now forms the island.

Along the head-race canal the upper layer is a cobble drift, several feet in thickness, probably a residuary ground moraine. Next below are cross-bedded outwash sands, having approximately horizontal beddings; many of the layers, however, are lenticular in form. The sand varies in size from moderately coarse to very fine, and "ripple" marks were frequently uncovered. Beddings of fine sand, generally dark in color, containing some aluminum and resisting water were found at vertical intervals.

At the power-house site, the upper layers of fairly fine to coarse sand are underlain by very fine deposits, which are in the nature of silt and contain enough clay to change entirely the behavior of the material. Beneath these fine sandy silts, the layers of which are impervious when undisturbed, is a deposit of very fine sand of apparently great depth, as exploration borings did not reach the bottom. Beneath this fine deposit of almost pure quartz sand must be a firm ground moraine. The materials at the power-house site, as outlined by Dr. Charles P. Berkey, the geologist, are sketched diagrammatically in Fig. 4 on which is also indicated the power-house, the intake dam at the forebay, and the penstocks.

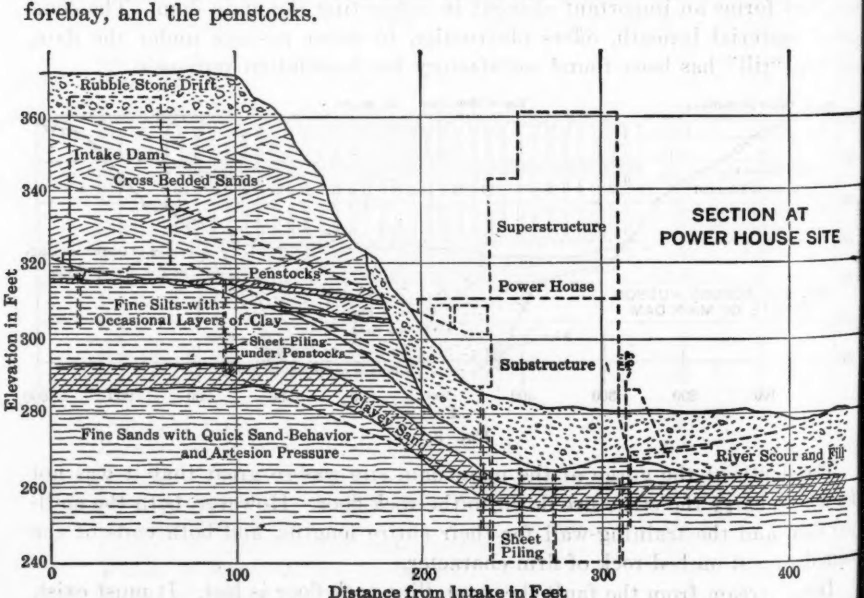


FIG. 4.

The depth and character of the fine sand at the power-house site prevented the use of any substructure supports. In consequence, the base was kept above the fine sand layer and did not penetrate (except the steel sheet-piling) the blanket of sandy silts and clays, which cap the finer materials. As the borings

which entered the "fine sands with quick behavior" exhibited an Artesian pressure, it seemed evident that the blanket or capping of clayey sand was practically impervious, and that the fine sands would not be released unless disturbed. The Artesian effect was due to the general water-table standing in the bank at elevations above the river level, thus furnishing a "head".

After this Artesian pressure was noted, a search was made for a more favorable power-house site, but all the territory in the neighborhood was found to be of similar character.

Hydrology

The drainage area above Sherman Island is 2 782 sq. miles. The region is mountainous, part of it heavily wooded, and contains many lakes and

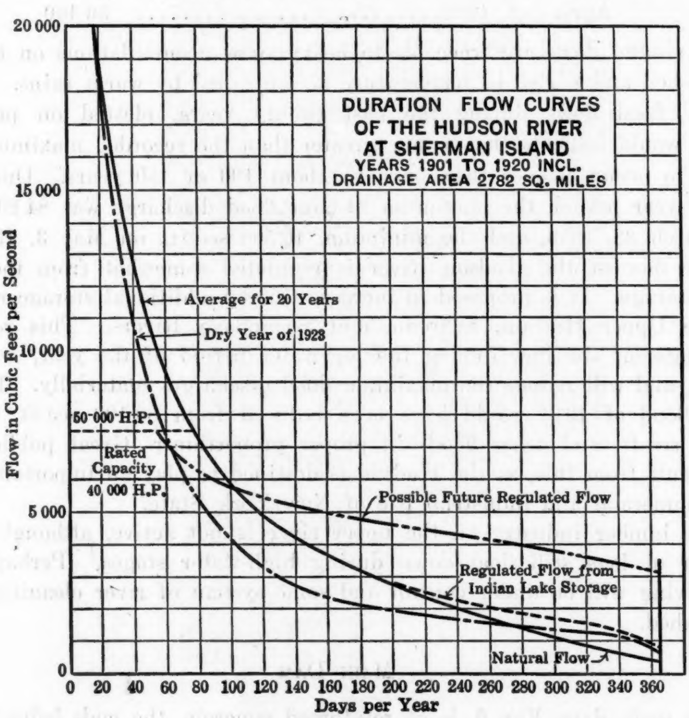


Fig. 5.

ponds. The ground is rocky or only lightly covered. The principal tributaries above the dam site are the Indian, Schroon, and Sacandaga Rivers. The average rainfall on the Upper Hudson water-shed probably exceeds 42 in. per year. The U. S. Geological Survey has recorded the flow for many years at gauging stations at Hadley and at Spier Falls. The normal flow, being an average of twenty years, 1901 to 1920, inclusive, is shown in Fig. 5, together with the regulation from Indian Lake storage, and, by way of comparison, the minimum flow during the dry year of 1923.

The greatest known flood, 89 100 sec-ft., was recorded on March 28, 1913. For the last eleven years, the maximum discharges* at Spier Falls (or Hadley) have been, as follows:

Date.	Discharge, in second-feet.
March 28, 1913	89 100
April 20, 1914	52 000
April 13, 1915	26 600
May 19, 1916	28 000
June 12, 1917	38 100
April 4, 1918	34 500
April 13, 1919	32 000
April 1, 1920	29 000
March 22, 1921	32 800
April 13, 1922	58 000
April 8, 1923	36 400

Maximum flows are traceable to heavy snow accumulations on the upper water-shed and a rise in temperature accompanied by warm rains. A study of the flood flows during the past twenty years, platted on probability paper, would indicate that a flow greater than the recorded maximum is not likely to occur oftener than once in about 100 or 150 years. During this twenty-year period, the maximum 24-hour flood discharge was 84 200 sec-ft., on March 28, 1913, and the minimum, 16 500 sec-ft., on May 3, 1911.

The flow of the Hudson River is regulated somewhat from the Indian Lake storage. It is proposed to increase this by additional storage reservoirs on the Upper Hudson, Schroon, and Sacandaga Rivers. This regulation will augment the flow during the low-water period of the year, as seen in Fig. 5, and will reduce the maximum flood discharges materially. Thus, the great flood of 1913 would have been reduced from 89 100 sec-ft. to about 55 000 sec-ft. and other floods in proper proportion.† Great public benefit will result from this, as the Hudson is destined to play an important rôle in the commercial and industrial life of New York State.

The lumber industry on the upper river is not active, although a large number of logs still float down during high-water stages. Perhaps, later, log driving will be under control and some system of river cleaning will be established.

Main Dam

The main dam, Fig. 6, is of reinforced concrete, the ends being on rock and the central portion on a sand and boulder deposit. The south end butts against the spillway dam, and the north end forms a wing-wall, which joins the head-works built at the entrance to the head-race canal (Fig. 2). The dam is straight, although the writer's original plan contemplated a slight camber up stream. It is a multiple-arch structure, having thirty-one bays, each 19 ft. center to center of buttresses. The length, including the north wing-wall, is 725 ft. 6 in., and the maximum height, 80 ft. 9 in., as

* Reports of New York State Engr., and U. S. Geological Survey.

† Report on the Water Power and Storage Possibilities of the Hudson River, New York Water Power Comm., 1922.

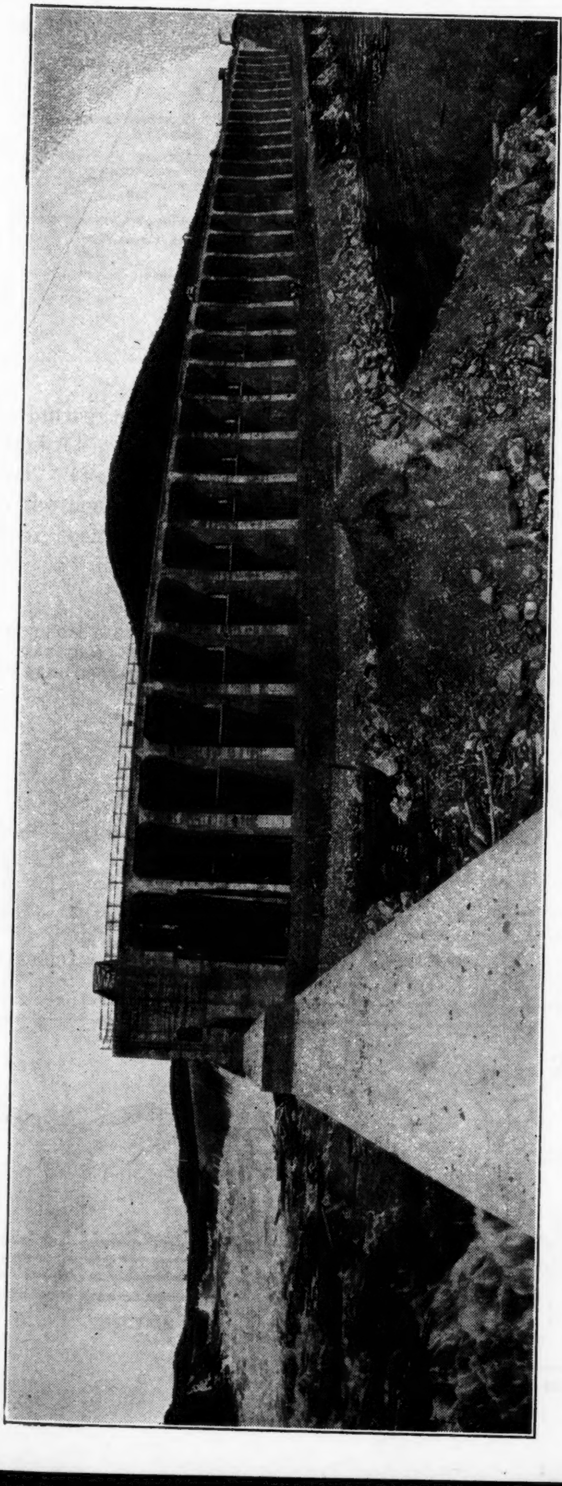


FIG. 6.—VIEW OF SHERMAN ISLAND DAM AND WASTE-WAY FROM DOWN STREAM.

measured from the under side of the central rib at the lowest part to the walk on top.

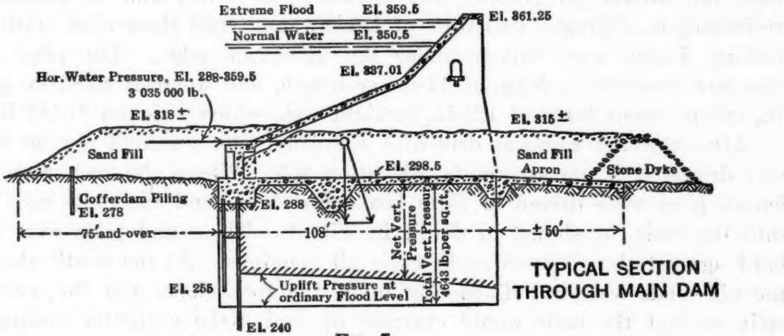


FIG. 7.

The buttresses, 3 ft. 6 in. thick, stand on a concrete base, 108 ft. wide by 3 ft. thick (Figs. 7 and 8), which extends from rock wall to rock wall across the river. The lower portion of the up-stream face is inclined 22° 40′ (5 on 12) to the horizontal, and the upper portion, 45 degrees. At the up-stream corner or heel, openings 10 ft. high by 13 ft. 6 in. wide were left in each bay (Fig. 8) for passing the river prior to closing the dam.

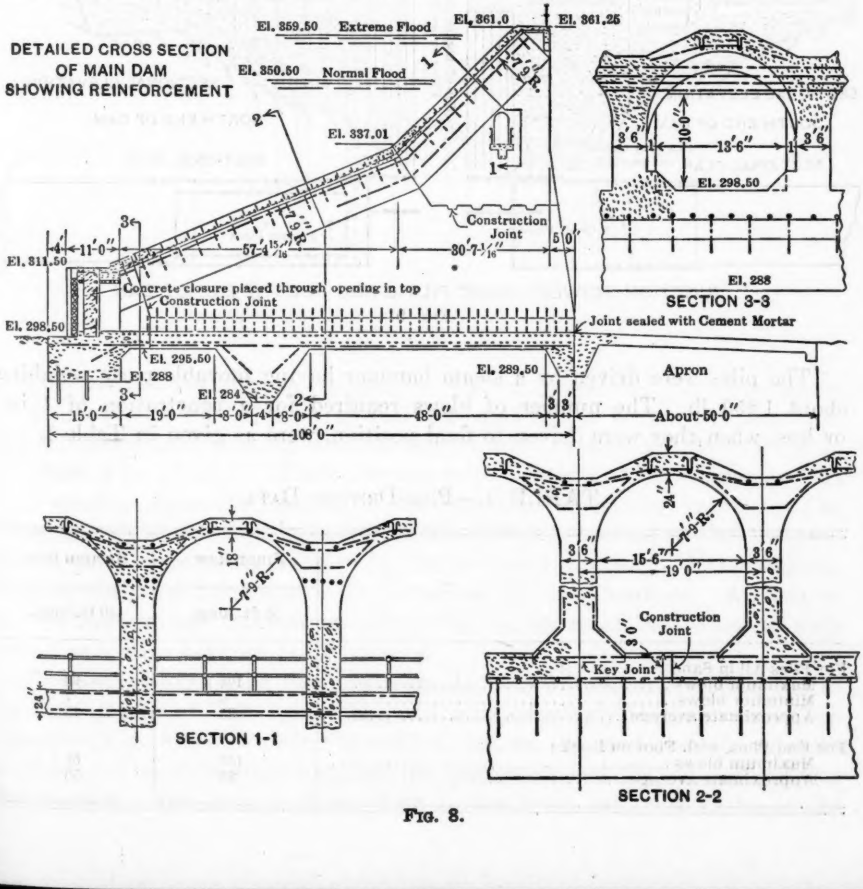
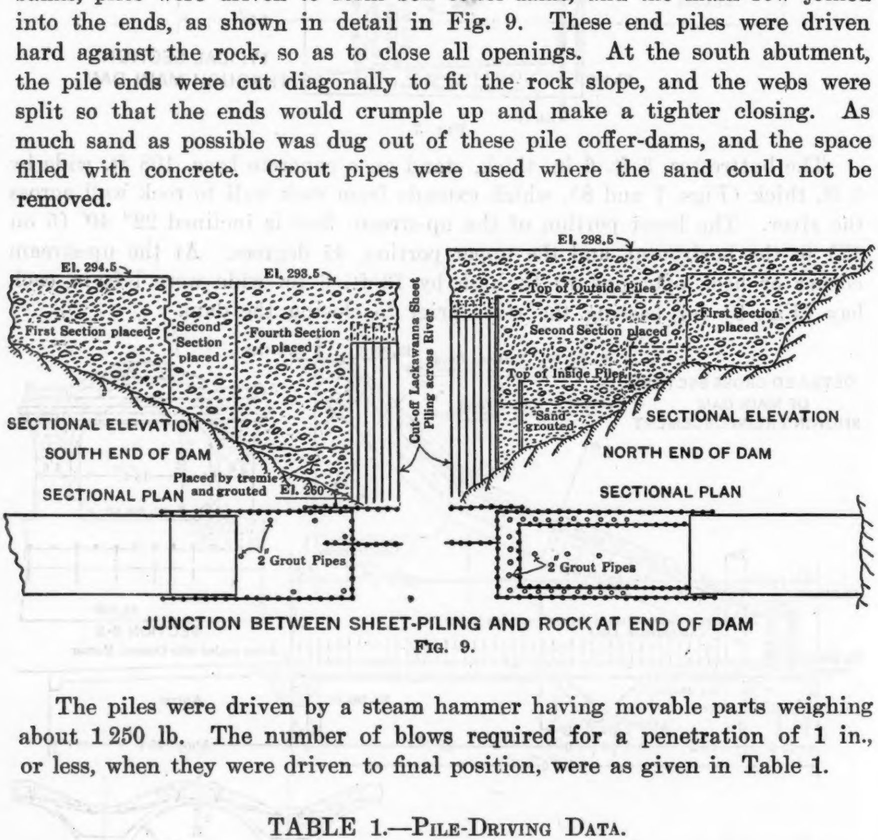


FIG. 8.

There are three concrete ribs under the base at the toe, the heel, and near the middle to prevent any tendency to scour, and to increase the resistance to sliding. Two rows of Lackawanna steel sheet-piles, with interlocking joints, were driven under the up-stream edge. The piles in the first row were 55 ft. long, of 14-in. arch web, and weighed 40.83 lb. per lin. ft., except about forty of 12¾-in. straight web, which weighed 37.187 lb.

After the first row was driven to Elevation 240, a second row 40 ft. long was driven 6 ft. up stream to Elevation 255. Where the rock rose at the banks, piles were driven to form box coffer-dams, and the main row joined into the ends, as shown in detail in Fig. 9. These end piles were driven hard against the rock, so as to close all openings. At the south abutment, the pile ends were cut diagonally to fit the rock slope, and the webs were split so that the ends would crumple up and make a tighter closing. As much sand as possible was dug out of these pile coffer-dams, and the space filled with concrete. Grout pipes were used where the sand could not be removed.



JUNCTION BETWEEN SHEET-PILING AND ROCK AT END OF DAM

FIG. 9.

The piles were driven by a steam hammer having movable parts weighing about 1 250 lb. The number of blows required for a penetration of 1 in., or less, when they were driven to final position, were as given in Table 1.

TABLE 1.—PILE-DRIVING DATA.

	FIRST ROW.	SECOND ROW.
	55 ft. long.	40 ft. long.
For Piles All in Sand:		
Maximum blows	104	92
Minimum blows	6	7
Approximate average	25	15
For End Piles, with Foot on Rock:		
Maximum blows	150	66
Approximate average	25	50

At the start, each pile was driven singly, with the first pile near the middle of the row; afterward, they were driven in "gangs", and greater success was obtained in keeping them straight and in lock. When a "gang" of, say, ten or a dozen piles was partly driven, another "gang" was set up and driven. Then, the first "gang" was driven a little more. This operation continued until the piles were down to the required elevation. Special wedge-piles were driven as required, to keep the row straight and vertical.

Before pouring the base concrete, all loose surface material was removed from the river bed, but the solid packed portion and boulder layer were disturbed as little as possible. Under the south portion of the dam the base was made at a lower elevation in order to fit the contour of the river bed. The base was reinforced, the rods projecting across the construction joints (Fig. 10). Where the base passed from rock to sand foundation, the rock was trimmed or cut, and a tapering sand cushion placed so as to ease the transition and minimize settlement troubles as much as possible.

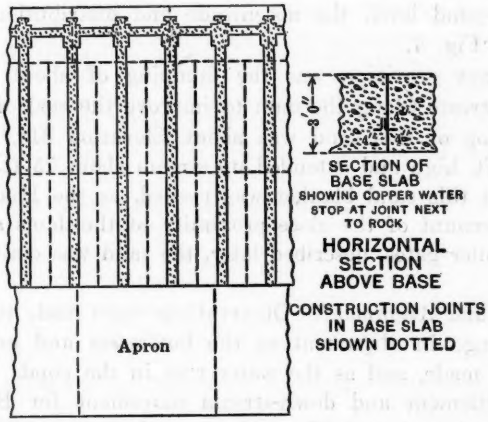


FIG. 10.

The buttress concrete was poured continuously between the construction joints shown in Fig. 8, leaving the arch decking to be placed afterward. The buttresses and decking were reinforced with deformed rods, varying in size from $\frac{1}{2}$ in. to $1\frac{1}{4}$ in. The buttresses were located at 19 ft. centers, which close spacing made the loading on the base slab practically uniform. The comparatively short span, moreover, facilitated the repeated use of the arch forms, which were easily handled by the overhead cableway. For simplicity in form work, the thickness of the buttresses was kept uniform. An inspection gallery extends through the dam (Fig. 8), its reinforced concrete walk acting as a strut between the buttresses.

The arched barrels of the up-stream decking are circular with a radius for the intrados of 7 ft. 9 in., measured perpendicularly to the slope. Each extrados (Fig. 8), was made three-sided, flat on top, and inclined on both haunches. This flattening simplified the form work and the labor of pouring the concrete. The deck is 24 in. thick in the lower part, and 18 in. in the

upper. Before the dam was closed, the northern half was given four coats of "Ferritex" water-proofing.

The arched deck had a flat incline so that the superimposed weight of water would increase the resistance to sliding. To add still further stability against sliding, the interiors of the bays between the buttresses were partly filled with sand. For each 19-ft. bay the water on the dam to the extreme flood level weighs 3 004 700 lb., the concrete, 4 272 000 lb., and the sand fill within the dam (exclusive of the apron), 2 250 000 lb., making a total of 9 526 700 lb. This weight gives an average loading of about 4 643 lb. per sq. ft. The resultant cuts the bottom of the base about 1½ ft. from the center. Under normal water conditions, the resultant cuts the base slightly on the up-stream side of the center, while for ordinary flood levels, it is practically at the center and produces a uniform loading. This downward pressure is reduced by the uplift, which was measured by test pipes built through the base, as described hereafter. Taking actual test readings for ordinary flood-pond level, the magnitude and distribution of the pressures are shown on Fig. 7.

An important provision was the dumping of about 30 000 cu. yd. of fine sand up stream along the dam to improve the seal of the cut-off sheet-piling. The top of this sand was about Elevation 318, that is, the dump was about 20 ft. high and extended up stream about 75 ft., or more. It was considered that this sand blanket was needed, as the Hudson deposits little sediment on account of the close proximity of the dams above the site. In front of the roller gates, described later, the sand was omitted until the final closure.

Settlement and Alignment.—Observations were made to record the settlement and change in alignment as the buttresses and arches were poured, as the fill was made, and as the water rose in the pond. Table 2 gives the cumulative settlement and down-stream movement for Buttresses 6 to 26. End Buttresses 1 to 5 and 27 to 31 were wholly or partly on rock so that the movements of these buttresses were small.

The cross-piling between the north and south coffer-dams was under Buttress 12. When construction started on the south portion, the adjacent buttresses in the north portion were affected. Table 2 gives in brief the changing conditions. It will be noted in general that both settlement and down-stream movements increase gradually toward the center of the dam; the maximum settlement being 1.4 in. at Buttress 14, and the maximum horizontal movement, 1.63 in. at Buttress 18. The survey line for alignment was along the walk on top of the dam. Any uneven settlement or elastic deformation of the structure, therefore, would be reflected in the observed alignment.

Surveys taken since those recorded in Table 2 have shown little change in either settlement or alignment, and the dam appears to be at rest with the exception of changes due to temperature.

Apron.—To increase the length of water travel under the dam and as a precaution against scouring during construction, when the river was being

passed through the bay openings, the base was extended down stream as an apron 50 ft. wide (Fig. 7). Sand filling was placed on this apron to supply weight against uplift and to resist any sliding movement of the dam. A dike of broken stone protects the down-stream edge of the fill and concrete apron against wash from back currents created by the discharge from the waste-way channel.

TABLE 2.—Movement of Main Dam.

Date.	Progressive Settlement, in Inches.											
	South.			Buttresses.					North.			Remarks.
	6.	8.	10.	12.	14.	16.	18.	20.	22.	24.	26.	
October, November, December, 1921.	....	....	....	0.33	0.54	0.31	0.29	0.33	0.37	0.36	0.26	Construction in north coffer-dam.
September, 1922.	....	....	....	0.33	0.55	0.33	0.30	0.37	0.53	0.45	0.26	Concrete finished, Buttresses 13 to 30.
December, 1922.	0.05	0.00	0.00	0.57	0.64	0.52	0.44	0.42	0.53	0.45	0.26	North coffer-dam removed. South coffer-dam closed. Fill started under north arches.
February. 1923..	0.14	0.14	0.15	0.90	0.89	0.82	0.70	0.59	0.70	0.64	0.44	Pond raised to about Elevation 335.
March 1923.....	0.24	0.29	0.35	1.13	1.10	0.99	0.89	0.80	0.86	0.80	0.53	Pond raised to about Elevation 350.
June, 1923.......	0.40	0.45	0.45	1.24	1.40	1.29	1.08	0.90	1.05	1.00	0.76	Filling under arches completed.
September, 1923.	0.40	0.45	0.45	1.21	1.40	1.29	1.08	0.90	1.05	1.00	0.76	Pond had been over Elevation 354.

Lateral Movement, in Inches, After Dam Was Finished, but Before Closure, as Surveyed Along Walk on Top.

Date.	South.			Buttresses.							North.			Remarks.
	4.	6.	8.	10.	12.	14.	16.	18.	20.	22.	24.	26.	28.	
January 17, 1923.	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	Datum taken. Temperature 22° Fahr. Dam closed, March, 1923.
April 10, 1923....	0.13	0.36	0.61	0.74	0.74	0.74	0.82	0.90	0.99	0.86	0.73	0.62	0.24	Temperature, 52° Fahr.
April 19, 1923....	0.25	0.48	0.78	1.04	1.03	1.16	1.10	1.14	1.13	0.96	0.86	0.70	0.30	Sand filling nearly completed. Temperature, 61° Fahr.
April 29, 1924....	0.30	0.55	0.97	1.09	1.13	1.45	1.52	1.63	1.56	1.31	1.33	0.96	0.42	After one year. Temperature, 72° Fahr.

Spillway Dam and Waste Channel

The spillway dam has a total crest length of 864 ft. A maximum flood of 100 000 cu. ft. per sec. will cause an elevation of the pond surface of 8 ft. 9 in. above the crest. The top of the main dam would then be 2 ft. above the flood level without allowing for the flow through the head-race canal and the spillway sluice-gates. As the drainage area above the dam is 2 782 sq. miles, there is 1 lin. ft. of crest per 3.22 sq. miles. The area of the pond is 277 acres.

The spillway dam was founded on firm rock for its entire length. The ogee form in the writer's original design, shown by the dotted line in Fig. 11, was omitted in construction.

The dam was built in about 50-ft. sections, the concrete being poured in alternate sections with as few horizontal joints as possible. Right-angled grooves, $8\frac{1}{2}$ in. deep, were provided to act as keyways between sections. The concrete was not reinforced.

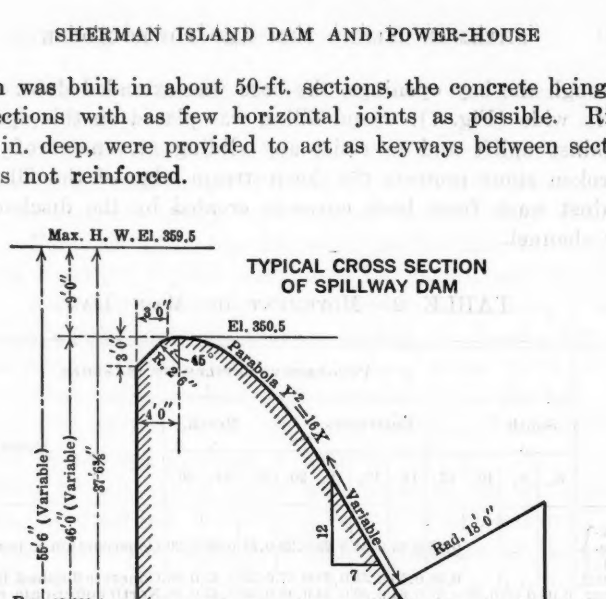


FIG. 11.

Brass pipes, 2 in. in diameter and 12 in. long, 5 ft. on centers, were placed along the crest, for the insertion of steel rods to hold flash-boards, 2 ft. high.

Seven gates to control the pond water were placed in a raised part of the spillway dam. Each gate is 13 ft. 4 in. high by 5 ft. 10 in. wide, made of 8 by 8-in. yellow pine timbers dressed and bolted to two 10-in., 54-lb. Bethlehem **H**-beams, which carry racks. The racks engage in a train of hand-power gears for raising and closing the gates. The wearing edges of the gates are protected by 4 in. by $\frac{1}{2}$-in. bronze strips and seat against 6 by 4 in. by $\frac{3}{4}$-in. steel angle-frames anchored to the concrete.

A concrete training wall extends down stream, so that the discharge from the waste-way may not cause scour or dangerous back eddies along the front of the main dam; it has a gravity section, without reinforcement, and is founded on rock.

Waste-Way Channel.—The waste-way, excavated in rock, is 172 ft. wide on the bottom opposite the end of the main dam. The bottom was roughly graded to a slope of 3.5%, increasing to 5% at the upper end. During extreme discharges, the velocity of the waste water will be about 31 ft. per sec. and its depth about 18 ft. when passing the end of the main dam. The design-

ing estimates for discharge and velocity were verified during the high water season of 1923.

Head-Works

The head-works at the entrance of the canal has a concrete base, built on rock and supporting nine reinforced buttresses, each 2 ft. 6 in. thick. The base contains a number of 1½-in. holes in each bay, behind the gates, to relieve any uplift pressure when the canal is empty. One end of the structure joins the wing-wall of the main dam and the other abuts the rock forming the opposite side of the canal.

An interesting feature is the design of the gates, which are ten in number, each 9 ft. 6 in. wide. Each gate has three sections, of steel frames filled with concrete, reinforced with rods welded to the frames. The lower and middle sections are joined by short links with the joints caulked; the upper and middle sections are connected with slotted links which permit the joints to open about 12 in. on lifting. Water can thus enter the canal to relieve the pressure on the gate and facilitate the raising of the lower part. Each gate weighs about 25 tons.

The gates are operated by overhead electric trolleys, and are controlled from the power-house for quick closing.

Head-Race Canal

The canal was dug in sand, except the first 150 ft. at the head-works end, which was in rock. The section is 32 ft. wide on the bottom (Elevation 324), with 1 on 1.6 side slopes to the ground surface at about Elevation 375. The excavation was made to approximately the proper section, with a longitudinal bottom slope of 1 ft. from the head-works to the intake dam. One-half the bottom concrete lining was poured in sections, 16 ft. wide, 42 ft. long, and about 8 in. thick. The other half of the bottom was used for a construction track to convey concrete from the mixer. After the first half was finished, the track was shifted to the concrete and the other half poured. The side slopes were then trimmed by hand scrapers to their true slopes, and the concrete was poured in rectangles 21 ft. long by 8 in. thick, to Elevation 346. From a construction joint at this point to the top of the lining, Elevation 356, the concrete was 3 in. thicker. The rectangles were poured alternately. Two timbers, each 6 by 4 in., were laid on the slope at each side of a rectangle to be poured. Between them was placed a No. 24 gauge (B. & S.) copper strip, 8 in. wide, crimped into a 1-in. loop, so that the copper water-stop would project 3 in. into each slab. The joints were brazed. The top forms, in 2-ft. sections, were of strong construction in order to support the concrete weights placed on them to counteract the uplift of the fresh concrete. The reinforcement, consisting of expanded metal of 3 by 8-in. mesh, weighing about 1 lb. per sq. ft., was laid on the finished grade, and pulled up by hooks into the proper position as pouring progressed. Only selected aggregate of small sized stone was used. Intermediate rectangles were laid in the same way, except that the inclined timbers were omitted, as the adjacent concrete slabs took the place of side forms.

The joints were filled with a bitumastic preparation known as Bridge cement. This was removed subsequently and the joints were grouted with a cement gun. The lining was sounded after it was all in place; where voids were indicated, holes were drilled and the grout was forced in under pressure. The mix of the grout was 0.1 part lime, 1 part cement, and 2½ parts sand.

After filling and quickly emptying the canal, weakness was disclosed in the lining at the down-stream end, due to ground-water and leakage, causing hydrostatic and earth pressure in the fine sand, some of which showed "quick behavior". For about 1 100 ft. (as also in the forebay), the lining was strengthened by building the north slope in steps. Concrete struts (Fig. 12) were constructed over the bottom slab of the canal. As the strutting was not adaptable to the forebay, the bottom slab then was covered to a depth of about 3 ft. with 2 500 cu. yd. of broken stone, to weight it against any uplift pressure and to oppose any inward thrust at the toe of the side slope lining. The broken stone was grouted with a cement gun in front of the racks to prevent dislodgment. As an additional precaution, flap relief valves of composition metal were placed about every 21 ft. longitudinally. These valves were 4 in. in diameter and mounted on pipes projecting 9 in. above the bottom concrete.

The canal lining was built to Elevation 356. At the forebay, the lining was built to Elevation 362, so as to protect the banks near the intake dam from surge waves.

At normal operation, with the water at the crest of the spillway, the cross-sectional area of canal above the cross-struts is 1 924 sq. ft. With five turbine units running at full gate, the water demand is about 7 500 cu. ft. The maximum velocity, therefore, is 3.9 ft. per sec.

In order to observe ground-water conditions as well as to detect any excessive leakage through the lining, eleven 2½-in. pipe wells were driven along the canal and forebay banks to about the elevation of the canal bottom; also, two additional wells, one on each side of the penstocks near the powerhouse. Observations of the water surface in the wells on the banks of the canal and forebay show only slight variations from the ground-water levels as determined during construction, indicating that leakage through the canal lining is small.

The water levels in the two pipe wells near the power-house show a variation of about 7 ft. These wells are at a much lower elevation than those along the canal or forebay, and this variation indicates changes in the ground-water at the lower level. This is probably effected somewhat by the obstruction to the flow of ground-water offered by the sheet-piling under the intake dam and by the penstock and power-house structures.

Intake Dam

The intake or forebay dam was founded in the sand of the plateau, as borings did not indicate rock within economical reach. After the sand had been excavated to grade, one row of Lackawanna interlocking steel sheet-piles was driven, as a cut-off, to a depth of 15 ft. below the bottom of the base slab; at the ends, where the wing structures tied into the sand, the

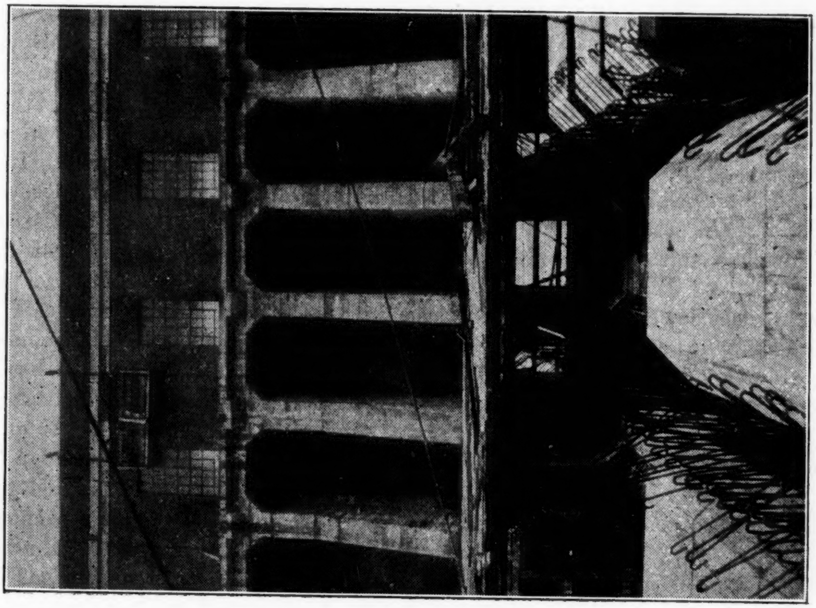


FIG. 13.—PENSTOCK FORMS, GATE-HOUSE IN BACKGROUND.

FIG. 12.—CANAL LINING STRUTS.

piles were stepped up. The total length, including the wing-walls, is 349 ft. 9 in., of which 200 ft. 6 in. is the intake dam. The height is 49 ft., the width on the top, 50 ft., and on the bottom, 55 ft. The base projects into the forebay about 5 ft., making the total width of the bottom slab about 60 ft. This projection was made to increase the stability when the stop-logs are in place and the water openings through the dam are empty.

This dam, Fig. 14, has fifteen openings for water, that is, three openings for each turbine unit. Each opening is 10 ft. 0 in. wide between the dividing buttresses, which are grooved for stop-logs and Broome gates. At the east end there are two concrete-filled, steel-frame gates, each 8 ft. wide, controlling the log and ice chute.

Racks, of 4 in. by $\frac{3}{8}$-in. steel bars, spaced $3\frac{1}{2}$ in. apart, protect each intake opening. The trash collected is raked into a reinforced concrete trough whence it can be flushed with water into the log and ice chute. Steam pipes from a heating plant near the power-house are carried to the racks, to free them from ice if occasion should require.

A brick gate-house on top of the dam contains individual electric motors and hoisting gears for the Broome gates. The dam is wide enough for a standard gauge track alongside the gate-house, which track connects the power-house with the railway to Glens Falls.

The weight of the intake dam and gate-house, including the equipment and water, is about 2 tons per sq. ft. of base area.

Settlement and Alignment.—Surveys were made at intervals to determine the settlement and changes in alignment, with results as given in Table 3. The survey plugs were located on top of the dam, a few feet from the down-stream edge as follows: Plug 1, at the west end of the west wing-wall; Plug 2 at the west end of the intake dam, about 65 ft. from Plug 1; Plugs 3, 4, 5, 6, and 7 equi-distant along the intake dam, about 38 ft. apart with Plug 7 at the east end; and Plug 8 at the east end of the east wing, about 80 ft. from Plug 7. Plugs 1 and 4 were disturbed during construction.

Table 3 supplies the information required to study the actual movement of the dam. When the canal is empty, the resultant pressure is about 2 ft. from the center on the forebay side; when the canal is full and the gates are closed, it is still on the forebay side of the center. Under operating conditions, the resultant is practically at the center. As the survey plugs were near the down-stream edge of the top of the dam, any slight tilting of the structure would appear in the observations as horizontal movement.

Penstocks

Water is conveyed from the intake dam to the power-house through fifteen reinforced concrete penstocks. Each wheel unit is supplied by three penstocks, 10 ft. 8 in. wide by 10 ft. 0 in. high (Fig. 15), which unite into one conduit as they enter the scroll for the turbine wheel.

Reinforcing rods, 1 in. in diameter, and bent to fit around the corners, were set about 8 in. apart. Experience showed that concreting would have been easier had the vertical side-rods been made in two parts with ends lapped

at about mid-height. With the continuous rods bent over the top, it was somewhat difficult to pour and tamp the concrete on account of the close spacing of the rods.

TABLE 3.—MOVEMENT OF THE INTAKE DAM.

Date.	2.	3.	5.	6.	7.	8.	Remarks.
	Progressive Settlement, in Inches. Survey plugs.						
June 25, 1923....	0.096	0.096	0.156	0.132	0.060	0.120	Canal first filled. Part of sand on penstocks in place.
July 14, 1923.....	+0.024	+0.024	+0.072	+0.012	0.060	+0.120	Canal dewatered. September 1, 1923, canal refilled and sand on penstocks completed.
October 9, 1923..	0.036	0.180	0.168	0.156	0.060	+0.060	Canal full.
May 3, 1924......	0.036	+0.036	0.120	+0.108	0.012	0.084	Canal full.
June 20, 1924....	+0.156	+0.108	+0.048	+0.036	+0.048	0.024	Canal full.
....	+0.012	0.108	0.324	0.132	0.144	0.048	Net settlement.
	Lateral Movement, in Inches.						
June 25, 1923.....	N 0.108	N 0.072	N 0.108	N 0.132	N 0.036	S 0.120	Canal first filled. Part of sand on penstocks in place.
July 14, 1923.....	S 0.204	S 0.132	S 0.204	S 0.168	S 0.132	S 0.048	Canal dewatered.
September 1, 1923	N 0.060	N 0.072	N 0.096	N 0.060	N 0.072	0.000	Canal refilled and sand on penstocks completed.
April 29, 1924....	S 0.144	S 0.024	S 0.132	N 0.012*	S 0.204	N 0.012	Canal full.
June 20, 1924.....	N 0.048	0.000	N 0.036	N 0.012	S 0.024	S 0.017	Canal full.
....	S 0.132	S 0.012	S 0.096	.....	S 0.252	S 0.173	Net change in alignment.

NOTE.—The plus (+) signs indicate a rise in the plugs due to the change in conditions mentioned under "Remarks." "N" indicates a movement north toward the forebay, and "S" a movement south toward the power-house.

* Plug probably moved.

The reinforced concrete base was laid directly on the sand, after the excavation, made by a steam shovel, had been graded by hand. A single row of interlocking steel sheet-piles was driven along each side, and also across the base above the highest visible ground-water. These piles were about 25 ft. long, and were intended to cut off the ground-water flowing under the penstock structure. The pouring of concrete (Fig. 13) was commenced at the power-house end, so as to work uphill. Wooden forms for the tube work were made in collapsible sections, and were kept in place for more than a week before removal.

The penstock structure was not tied into the power-house base or into the intake dam, but all construction joints had a $\frac{1}{16}$-in. lead sheet built in as a water-stop and the joints caulked with oakum from the inside. The interior surfaces of all conduits were coated with "Ferritex" after the concreting was finished.

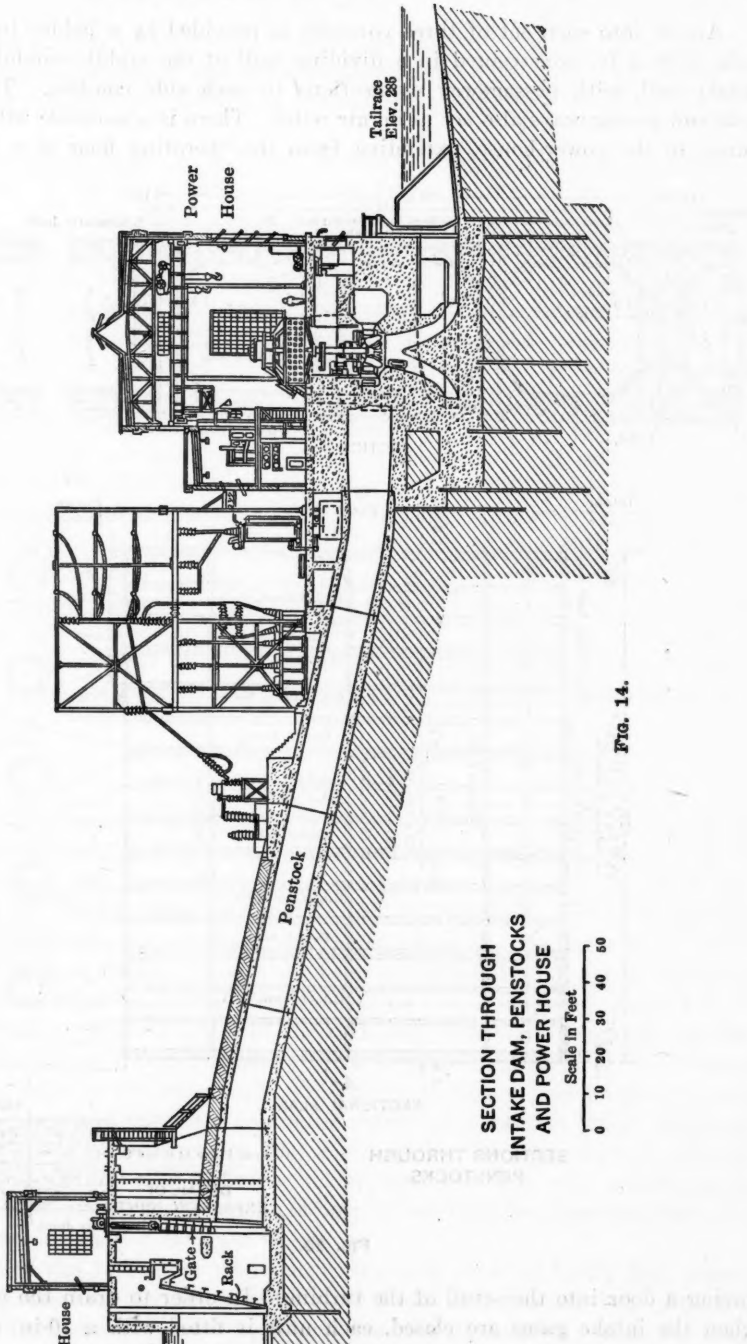

Gate House
Elev. 350
Intake
Gate
Rack
Penstock
Power House
Tailrace
Elev. 285
SECTION THROUGH
INTAKE DAM, PENSTOCKS
AND POWER HOUSE
Scale in Feet
0 10 20 30 40 50

FIG. 14.

Access into each set of three conduits is provided by a ladder in a manhole, 2 by 2 ft., constructed in a dividing wall of the middle conduit at the intake end, with passageway connections to each side conduit. The manhole and passageways also act as an air relief. There is a manhole with ladder rungs in the power-house, extending from the operating floor to a chamber

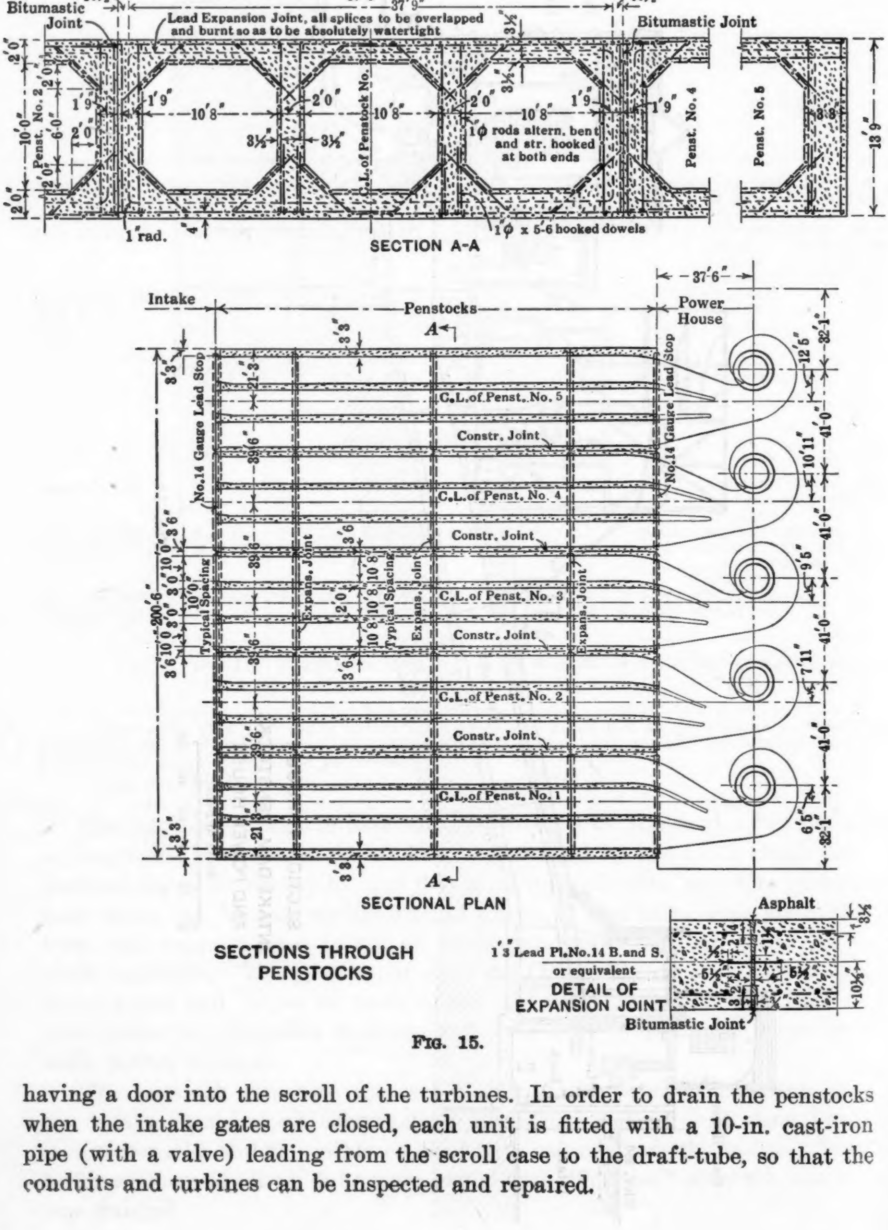


FIG. 15.

having a door into the scroll of the turbines. In order to drain the penstocks when the intake gates are closed, each unit is fitted with a 10-in. cast-iron pipe (with a valve) leading from the scroll case to the draft-tube, so that the conduits and turbines can be inspected and repaired.

The lower part of the penstock structure supports concrete platforms for the outdoor transformers and lightning arresters. The upper part is covered with sand 2 to 3 ft. in thickness to reduce the temperature range as much as possible.

The penstock structure is 176 ft. in length and has five cross-construction joints, one at the intake end, one at the power-house wall, and three intermediate. When the tubes were filled with water, then emptied, and refilled, a slight vertical movement was observed at the cross-joint about 35 ft. from the power-house. After a few months' use, this fluctuation seemed to cease. When the pressure on the power-house substructure from water in the penstocks was first applied, there was an initial movement, causing leakage at the joint between the power-house and the penstocks. In order to reduce this leakage to a minimum, crimped, 37-oz., copper strips, 36 in. wide by 14 ft. long, with edges brazed, were fastened across the joint. These copper strips were held between a pure rubber gasket, $\frac{1}{2}$ in. thick by $2\frac{1}{2}$ in. wide, next to the concrete, and a wooden strip, 1 in. thick by 6 in. wide, on top of the copper, by $\frac{5}{8}$-in. brass expansion bolts and 3-in. washers. The crimping allowed for an expansion of 2 in. In each tube there was a $2\frac{1}{2}$-in. flap relief valve, brazed to the copper, opening inward, to relieve the external water pressure and prevent the copper from tearing, should the tube be empty. This remedy was successful in bringing the leakage under control. Continuous movements of the joint, as measured by a wedge and a micrometer after the initial movement occurred, indicate that the opening of the joint varies with changes in temperature.

Power-House

The power-house is 228 ft. long by 69 ft. wide, outside dimensions. Water passages and working galleries are built in the reinforced mass concrete of the substructure. The superstructure is of brick, trimmed with belt courses of precast concrete blocks. The steel roof trusses, gallery floor, and traveling crane are supported on steel columns. All the windows and skylights are of wired glass set in steel frames. The main floor was finished with a 1-in. surfacing after the electrical machinery was in place; its area is 15 742 sq. ft., or 1 sq. ft. for each 3.2 rated horse-power of the turbines. The traveling crane, electrically operated, has a lifting capacity of 50 tons and a span of 50 ft. Heating is accomplished by two steam coils, equipped with blowers, discharging through ducts opening into the main generator room; but operation shows additional heat to be seldom required, as that given off by the generators is sufficient to warm the building in ordinary winter weather. The total weight on the foundation of the power-house, including its substructure, superstructure, and equipment, averages 3 tons per sq. ft.

The site was in shallow water near the north shore of Little Bay, a point of land protecting the house from river floods and floating ice. An earth dam was built enclosing the site, which was then de-watered. After the house and tail-race had been finished, this dam was removed by steam shovels and draglines.

The sand deposit at the power-house site was of such nature that great care was required during construction. After information collected by test borings was studied, a well was dug inside a steel sheet-pile casing, 9 ft. in diameter (Fig. 16). A dark fine sand layer containing some clayey characteristics was reached at about Elevation 265. This was the blanket or impervious layer previously mentioned, which overlaid the fine sand with "quick behavior" that had an Artesian pressure due to ground-water. A test pit was then arranged in the well; the piston, having an area of 1.737 sq. ft., was loaded with concrete blocks, and the settlement measured (Fig. 16). Under a weight of 8.7 tons per sq. ft., after 50 hours, no further settlement of the piston occurred. This loading was 2.9 times the unit weight of the completed building.

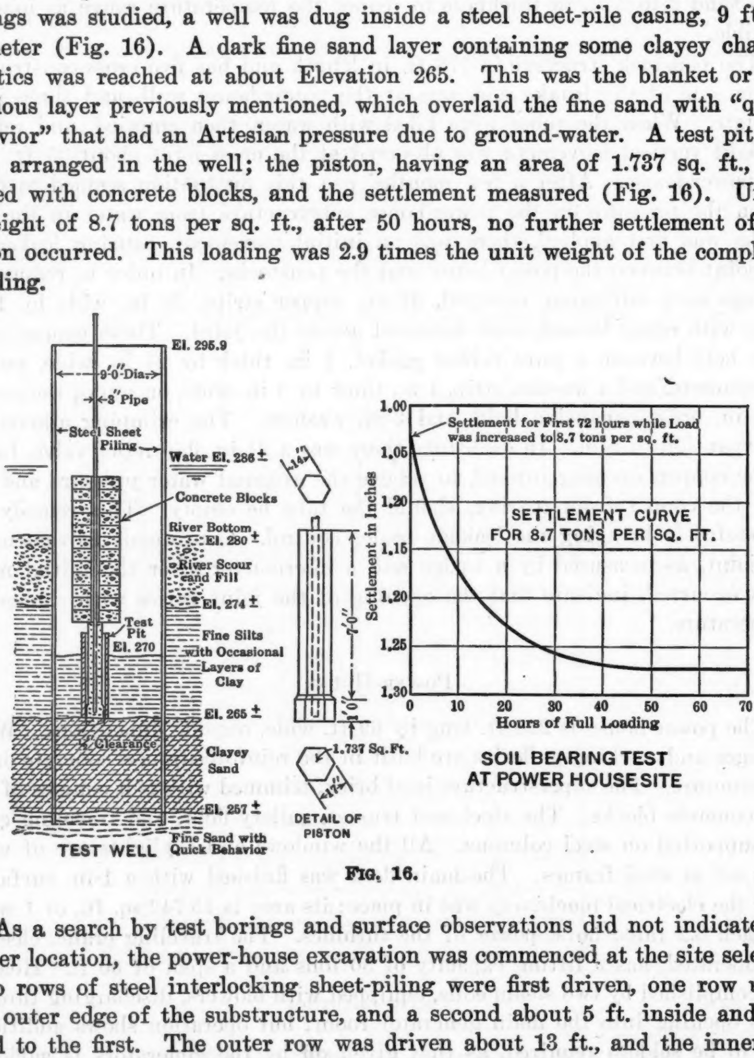


FIG. 16.

As a search by test borings and surface observations did not indicate any better location, the power-house excavation was commenced at the site selected. Two rows of steel interlocking sheet-piling were first driven, one row under the outer edge of the substructure, and a second about 5 ft. inside and parallel to the first. The outer row was driven about 13 ft., and the inner row about 30 ft., below the bottom elevation of the concrete base. Then cross-rows were driven in both directions, dividing the area into ten rectangles about 41 by 35 ft. in size.

When the work began, it was evident that in the wet excavation great difficulty would be experienced in holding the bank on the north side across a wide excavation, without an equivalent burden of sand on the south side. The flat slope of repose of the fine wet sand in the water-bearing strata of the north bank made it expedient to disturb the bank as little as possible. These conditions dictated the next step, which was to drive a third row of

sheet-piles along the north side next to the bank, about 21 ft. inside the outer row, and to excavate sections between these rows. When the desired sub-grade was reached, the sections were filled with concrete to Elevation 284. This concrete served temporarily as a retaining wall against the sand bank, while the excavation for the remainder of the substructure was being made.

Then the spaces between the two outer rows of sheet-piling around the other three sides of the power-house site were excavated to grade and filled with concrete, following which, the interior rectangles were excavated by a clam-shell bucket, one after another, until there was risk of creating boils of dangerous size through which the fine sand from below might flow. This risk was indicated by the beginning of small boils in isolated localities. When these small boils gave evidence of increasing, the holes were filled with sand and broken stone. This expedient was successful; and the excavation was carried to the desired elevation, namely, about 15 ft. below the original sand bottom, the water being kept under control by pumping from sumps in neighboring rectangles. The sumps were filled with broken stone which acted like filters and minimized the danger of pumping out sand. While the sump pumps kept the water under control, a sub-base of concrete, 12 in. thick, was deposited in the rectangles by self-dumping buckets. This prevented further boils. On top of the sub-base, the substructure concrete was poured in alternate rectangles, to keep the loading on the sand as uniform as possible over the area. All irregularities in settlement were corrected by bringing the concrete in the various rectangles to uniform levels as the work progressed. Cross-braces between the rows of sheet-piling were removed and the sheet-piles burned off at the time the concrete was carried up to the level of the bottom of the draft-tubes.

The settlement of the structure was measured as the work progressed. The greatest settlement, caused partly by pumping, was $2\frac{11}{32}$ in., near the south-east corner, of which 2 in. occurred in the sub-base before the substructure concrete was poured. The least settlement was $1\frac{3}{32}$ in. and the whole averaged about $1\frac{15}{32}$ in., being fairly uniform, except at the one corner. Subsequent surveys showed $\frac{1}{8}$ in. additional settlement, when the superstructure was erected, after which no further settlement was noted. As previously mentioned, when the penstocks were first filled, the water pressure on the power-house caused a slight horizontal movement.

The draft-tubes are annular, of the Moody spreading type, with a central concrete cone, the armored apex of which is 5 ft. below the bottom of the runner. The discharge into the river from each wheel is through two openings, each 15 by 11 ft. (Fig. 14).

Considerable vibration occurred as each unit operated through a range between 3 500 kw. and 4 500 kw. at ordinary tail-water level. This vibration was traced to the formation of a vacuum between the apex of the concrete cone and the bottom of the runner. The trouble was remedied by inserting a 4-in. pipe through a stuffing-box fastened to the manhole cover of the inspection gallery. The pipe extended about 18 in. into the draft-tube, and was fitted with a coiled spring foot-valve at the outer end in the gallery, and

a vacuum gauge and stop-valve at about mid-length. By operating a unit through the full range of gate opening and by manipulating the stop-valve while observing the vacuum gauge, the foot-valve was adjusted to open and close automatically within the desired range of vacuum. This simple device, suggested by C. A. Terry, Superintendent at the power-house, worked well, and effectively stopped the vibration and water-hammer.

Tail-Race.—Water discharged from the turbines passes directly into the river. The elevation of the bottom of the discharge openings is about 9 ft. below the natural bed of the river, and, therefore, the area was excavated to grade. As the discharge channel widened, the depth of cutting was reduced until the natural level of the river bed was again reached. A concrete pavement to prevent scouring (Fig. 14), was laid along the discharge front of the power-house, extending out 50 ft., and inclining upward on the gentle slope of the excavation. For the first 20 ft., this slab is 30 in. thick and is reinforced between the buttresses of the turbine discharge passages, which buttresses extend out on the slab to resist any uplift pressure. For the remaining 30 ft., the slab is thinner and unreinforced.

Log Chute

As the river bed below the main dam is practically dry when water is not passing over the spillway, a logway was built in the head-works to float logs, whenever necessary, into the canal. A chute was built in the intake dam to pass these logs and excessive ice accumulation into the river.

This chute consists of openings in the intake dam, each 8 ft. wide by 9 ft. high, closed by concrete gates, which connect with two passages through the dam. The two passages unite at the discharge portal into a timber log chute, which extends to the river at the east end of the power-house.

Closure of River

As a start in construction operations on the main dam Lackawanna steel sheet-piles, to form the first coffer-dam, were driven from the north bank to the center line of the bay between Buttresses 10 and 11. These piles were about 35 ft. long, with tops at Elevation 317, and penetrated the sand about 18 ft. The rows were braced against log cribs, 10 by 30 ft., filled with stones. The interior was de-watered by motor-driven centrifugal pumps. The top layer of loose material was removed, and trenches were dug for the cut-off ribs. The concrete base was then laid, and the buttress forms were set up and poured, beginning at the north end.

A cross-row of piles was driven on the center line of Buttress 12, joining the up-stream and down-stream rows. The tops of these cross-piles under the dam were cut off to end in the concrete base of the dam, while those on both the up-stream and down-stream sides of the structure projected to Elevation 317. This cross-row formed the north end of the second coffer-dam for constructing the south portion of the dam.

In the beginning, the river flowed through the channel left between the end of the first coffer-dam and the south bank. After the north portion of the dam was built, the up-stream coffer-dam piling was burned off at about

Elevation 302, during low water, and the down-stream piling was pulled. The river then was allowed to flow through the bays between the buttresses in the north portion, while the second coffer-dam and south portion of main dam were being built.

When the dam was completed across the river, the up-stream and down-stream sheet-piling of the second coffer-dam were burned off, the down-stream row at about Elevation 301. Stop-logs were placed in grooves at all the bay openings except four, namely, between Buttresses 5 and 6, 6 and 7, 8 and 9, and 9 and 10, and mass concrete was poured between the stop-logs and bulk-head forms at the upper end of the bays. The river then flowed through the four bays left open, and in case of emergency there were the eleven openings in the spillway dam. Of these latter openings, four were temporary, each 9 ft. wide, and seven were permanent, each 5 ft. wide, closed by wooden sluice-gates.

When the time arrived finally to close the last four bay openings, roller-gates (Fig. 17) were constructed, similar in principle to those used by M. Caméré in France. The rollers were steel pipes, 36 in. in diameter, filled with concrete and old iron, so that they would sink by their weight. A curtain of $7\frac{1}{2}$ by $7\frac{1}{2}$-in., dressed yellow-pine timbers, hinged together by 1-in. plow steel rope, was wrapped around the central pipe core, with one end fastened to the dam at the top of the opening and the other end to the core. Cables of $1\frac{1}{4}$-in. wire rope were wrapped around the projecting ends of the core, and carried to a windlass on the dam. A heavy canvas covering was placed over the timbers on the up-stream side to make the dressed joints as tight as possible. Adjacent sides of the logs were slightly beveled, so that their faces would fit tightly together. The roller-gates were lowered and raised a number of times before their final closing.

When all was in readiness, the four roller-gates were lowered in quick succession, and concrete was placed behind them in the bays, by chuting it through 36-in. steel pipes, the upper ends of which were above the temporary pond level. The river then passed entirely through the openings in the spill-way dam into the waste channel.

The temporary spillway openings were stopped by logs and filled with concrete, leaving the river to pass through the seven permanent gate-openings. These gates were closed as the final act, and the water rose in the pond until it passed over the crest of spillway.

Pressure Test Pipes

The original design contemplated thirty-nine test pipes, three in each alternate bay, evenly spaced up stream and down stream. The pipes were 3 in. in diameter, of extra heavy wrought iron, shod with a point, and perforated with 160 holes, $\frac{1}{8}$ in. in diameter, for a distance of 10 ft. just above the point. These pipes extended under the base slab, about 10 ft. into the sand. Each pipe was to have a composition cap, tapped for a $\frac{1}{2}$-in. extension. As constructed, seventeen pipes were placed, with four bays having

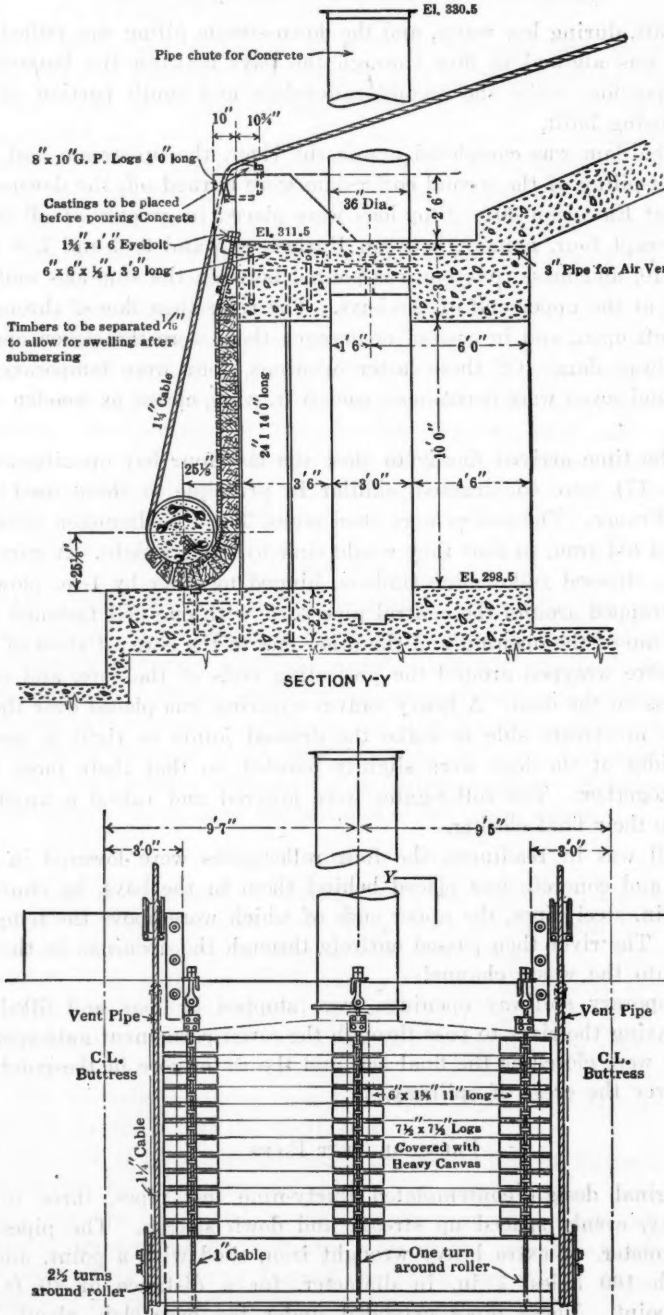


FIG. 17.—ROLLER CURTAIN GATE.

two pipes and three bays, three pipes each, the up-stream and down-stream spacing being irregular.

The uplift pressure was measured by the height of the water standing in these pipes. It was found that the pressures were not the same in all the bays, but, in general, were greater under the northerly portion (where the sand in the river bed was finer) than under the southerly portion.

Although the uplift pressure is a function of the total head, as measured between water elevations above and below the dam, its magnitude varies with temperature and other conditions. The elevation of the water surface in the pipes varies closely with changes in elevation of the river surface just below the dam, while changes in pond elevation have less relative effect.

Fig. 18 shows typical hydraulic grade lines through the dam. The average loss of head due to the sheet-pile cut-off is about 38 ft. The distance from the top of the sand fill above the dam to the bottom of the sheet-piling is about 80 ft., and the distance from the bottom of the piling to the bottom of the dam is about 50 ft., or a total of 130 ft. The average loss of head in passing through the sand and boulder foundation under the dam and apron is about 11 ft. in a horizontal distance of 150 ft.

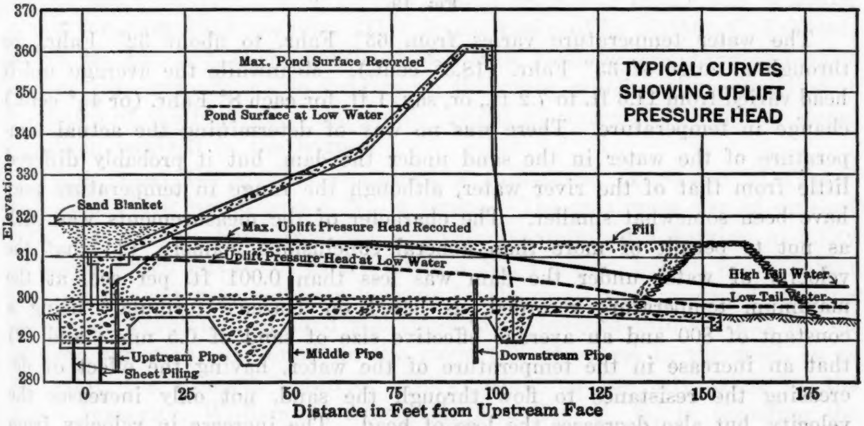


FIG. 18.

The average uplift pressure head (the average of all readings taken on the same day of the difference in water level in the pipes and tail-water) reached 11.3 ft. when the net head between the pond and tail-water levels was 50.5 ft., or about 22% of the net head created by the dam. The original calculations included an allowance of 26 per cent. Although the design considered the uplift pressure as acting on the entire base, the actual uplift pressure probably acts on less than 50% of the area, due to sand contact. The head as measured in the test pipes is practically the total head, as the velocity head would be negligible. The sheet-piling proved extremely efficient as a cut-off, considering the difficulty in driving it through the boulder layer. It would be well for other engineers to record their experiences with steel sheet-piling used as a cut-off under actual working conditions.

In Fig. 19 is illustrated an interesting relation between the seasonal water temperature and the average uplift pressure. Curves platted for individual pipes show the same characteristics as the average given in Fig. 19. The writer believes that this is the first record of the effect of temperature of water on uplift, as registered under actual conditions, for a large structure on sand. It will be noted that the difference in uplift between low and high temperature periods amounts to about 65% of the minimum uplift. Under other structures of similar character, it is possible that observed uplift pressures might cause anxiety unless the effect of temperature was considered.

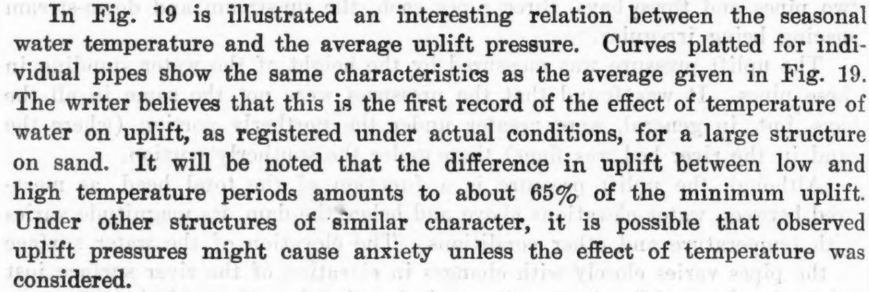


FIG. 19.

The water temperature varies from 65° Fahr. to about 32° Fahr., or through a range of 33° Fahr. (18.3° cent.). Meanwhile the average uplift head varied from 11.3 ft. to 7.2 ft., or, say, 1 ft. for each 8° Fahr. (or 4½° cent.) change in temperature. There was no way of determining the actual temperature of the water in the sand under the dam, but it probably differed little from that of the river water, although the range in temperature may have been somewhat smaller. The character of the measurements was such as not to permit of more than general conclusions, namely, (1) that the velocity of water under the dam was less than 0.001 ft. per sec. at the maximum temperature recorded, as based on Hazen's formula,* using a constant of 800 and an average effective size of sand of 0.5 mm.; and (2) that an increase in the temperature of the water, having the effect of decreasing the resistance to flow through the sand, not only increases the velocity, but also decreases the loss of head. The increase in velocity from the lower to the higher temperature was probably less than 60 per cent.

It will be noticed that the maximum point of the uplift curve for 1924 is less than that for 1923, although the maximum water temperatures were the same. This may be accounted for by a silting up of the pond and by a compacting of the sand blanket at the heel of the dam.

CONCRETE

For various parts of the work, the mix by weight was as follows, the required compressive strengths being for specimens 28 days old:

For arches, struts, and flood-gates (2 400 lb. concrete): 1 part cement to 6 parts aggregate (equivalent to 1:2:4); and for each 100 lb. of cement not more than 8 gal. of water.

* *Transactions*, Am. Soc. C. E., Vol. LXXIII (1911), p. 201.

For other reinforced concrete, such as the bottom slab, head-works, intake, power-house (2 200 lb., concrete): 1 part cement to 6.5 parts aggregate (equivalent to 1:2.17:4.33); and for each 100 lb. of cement not more than $8\frac{1}{2}$ gal. of water.

For buttresses and canal lining (2 000 lb. concrete): 1 part cement to 8 parts aggregate (equivalent to 1:3:5); and for each 100 lb. of cement not more than $9\frac{1}{2}$ gal. of water.

For the spillway dam, apron, training wall, and other mass concrete not reinforced (1 600 lb. concrete): 1 part cement to 9 parts aggregate (equivalent to 1:3:6); and for each 100 lb. of cement not more than 10 gal. of water.

Lime was added as follows: 6% for arch concrete; from 6 to 8% for the penstocks; 7% for the canal lining; but none for the buttresses, base slab, and similar concrete.

The cement used was Ironclad, made by the Glens Falls Cement Company, and shipped in bulk by railway to the site. Sand was selected from the better grades found in the excavations. A crusher plant furnished the broken stone, using selected rock taken from the excavations. Considerable trouble was experienced due to the hardness of the rock which damaged the crushers from time to time.

The main concrete mixing plant was equipped with a mechanical weighing device for controlling the mix. This device, designed by Nathan C. Johnson, Assoc. M. Am. Soc. C. E., consisted of a scale carrying the charging hopper on one side and adjustable weights on the other, which automatically shut the gates of the bins when the proper weight of cement and sand, or stone, had been received in the hopper.

Stone from the crusher contained a substantial proportion of "fines" and dust which would pass a No. 4 screen. The maximum size of crushed stone was $2\frac{1}{2}$ in. Sand from the excavations contained a varying quantity of gravel which would not pass through a No. 4 screen, but all gravel larger than $1\frac{1}{2}$ in. was removed by a "grizzly." Diagrams were prepared showing the proper proportion of sand and stone to be weighed into the mix for all possible variations of "fines" in the stone and of gravel in the sand, so as to obtain the strength required. Notwithstanding the care taken by the Chief Inspector, William A. Aiken, M. Am. Soc. C. E., in changing the proportion as indicated by mechanical analyses, it was found that uniform strength, with the same quantity of cement, could not be obtained until the "fines" were removed by the installation of a "dust jacket."

Bin cement was tested at the Glens Falls Laboratories of the International Paper Company, while cylindrical test pieces, cast in special pipe forms, 10 in. in diameter by 16 in. long, were sent regularly to the testing laboratories of the Rensselaer Polytechnic Institute at Troy, N. Y. As an added precaution, test pieces were frequently cut from the finished work and sent to the laboratories.

For concrete mixed in freezing weather the aggregate was heated, and after depositing, the temperature was kept at about 50° Fahr., or more, for at least 72 hours, by using salamanders and by covering the surfaces. All important forms were coated with oil and wetted before pouring the con-

crete. Forms were kept in place for a minimum of five days, but when the daily temperature fell below 70° Fahr., the minimum time was lengthened.

QUANTITIES

The following quantities were used in the finished structures, exclusive of structural steel and cement for grouting and "guniting":

Rock excavation	159 899	cu. yd.
Sand and earth excavation	799 160	" "
Trimming canal slopes	68 600	sq. yd.
Sand and rock-fill	118 352	cu. yd.
Rip-rap	11 870	" "
Concrete	127 803	" "
Pre-cast concrete trim	4 849	lin. ft.
Reinforcing steel	1 973	tons
Expanded metal	291	"
Steel sheet-piling	2 335	"
Brick	826 790	
Expansion joints	43 405	lin. ft.

In addition, considerable quantities of excavation, fill, lumber, concrete, and other materials were used for the falsework, camp, railroad, river cribs, and the like.

EQUIPMENT

The principal items of equipment were as follows:

One 50-ton Shaw over-head traveling crane, electrically operated.

Four turbines, William Cramp and Sons, I. P. Morris Department; head, 66 ft.; speed, 150 rev. per min.; 10 000 b.h.p.; vertical shaft; Kingsbury bearing (fifth turbine to be installed later).

Four main alternating generators (fifth generator to be installed later), with field switch panels; General Electric Type A. T. B., Form V; 48 poles; 9 000 maximum k-va.; 150 rev. per min.; P. F. 8; 7 200 kw.; at full load, 6 600 volts.

Four exciters, mounted on top of the alternators; General Electric direct-connected, Type M. C. P., Form A; 8 poles; 114 kw., maximum; 150 rev. per min.; 456 amperes; at full load, 250 volts; continuous 50° cent. rise.

One motor generator set, as extra exciter; induction motor, 1 E. 13 B, Form K; 3-phase; 60 cycles; 550 volts; 153 amperes; 1 170 rev. per min.; direct-connected to direct current generator, shunt wound, Type R. C. 37; 250 volts; 460 amperes; 1 170 to 1 200 rev. per min.

Four hydraulic governors, with Taylor Control System, William Cramp and Sons, I. P. Morris Department, Serial No. 152.

Three Woodward oil pumps to governors, link belt driven by induction motors, Type K T.; 3-phase; 40 cycle; 550 volts; 30 amperes; 765 rev. per min.

One air compressor for five accumulators.

One station switch-board; with control bench board and voltage regulators.

Seven subtractive polarity transformers; General Electric Type F. O., Form HDH.; volts, 6 600 to 114 500; cycles, 60; capacity, 7 500 k-va.; continuous 55° cent. rise; two banks of three; one spare.

Five high-tension oil circuit breakers; General Electric Type O. F. 1, Form B. 1, Class F. 2; maximum amperes, 400.

Three multi-whirl coolers, Griscom Russell Company; size, 111-60; shell, 50 in.; tubes, 50, driven by induction motors; General Electric Type K T., Form C; volts, 550; amperes, 14.7; speed, 1 760 rev. per min.

Eight oxide film arresters; alternating-current; General Electric Type OF, Form H. O.; volts, 92 400 to 115 500.

Transmission line cables; 7 wires, No. 0000; 114 000 volts.

Three lighting transformers; Type H, Form N.

Three station transformers; Type H; 100 k-va.; cycles, 60; volts, 6 900 to 11 950.

One set of storage batteries, with charging motor-generator set.

Twelve gate-hoisting gears (three more to be installed) for Broome gates at intake dam; Maine Electric Company; driven by induction motors, Type K T., Form B.; 3-phase; 60 cycle; 15 h.p.; 550 volts; 17.5 amperes.

Construction Plant

A standard-gauge, single-track railway, with sidings, was built over a leased right of way from Glens Falls to the site, a distance of about 7½ miles, with branch tracks to reach all parts of the development. Tracks were extended over bridges, both just above and just below the main dam, so that easy communication was had with the south bank. The bridge girders rested on the cribs supporting the steel sheet-piling of the coffer-dams.

Derricks were erected and shifted to serve all parts as the work progressed. A Lidgerwood cableway, steam-operated, was stretched over the center line of the main dam. The span was about 1 550 ft., the cable, 2½ in. in diameter, with a maximum allowable center load of 10 tons.

The stone crushers, cement storage, and principal mixer plant were on the north bank near the end of the main dam. Smaller mixers were located at about mid-length of the canal and at the forebay dam. Concrete was distributed by chutes radiating from towers, and also by dump-cars to those parts which could not be reached by the chutes.

A camp was built on the north bank, with completely equipped bunkhouses, dining room, cafeteria, kitchen, bakery, store, first aid hospital, machine shop, blacksmith shop, carpenter shop, etc. There was also an office building, with accounting and drafting-rooms for the engineers and contractor's staff, and a number of houses for the contractor's officers and their families, some of which were designed for permanent use by the Operating Staff.

A reservoir, for drinking water supply, was built on a neighboring hill, while an elevated water tank, supplied by a pump from the river, served for fire protection, construction, and sanitation. There was also a drainage system throughout the camp.

During active construction, the working force varied from 1 000 to 1 500 persons. Some employees resided in Glens Falls and were brought to and from the site by train, with free transportation.

Electric current for light and power during construction was supplied over a temporary transmission line from the Glens Falls Mill of the International Paper Company.

Personnel

A. H. White, M. Am. Soc. C. E., Chief Engineer of the International Paper Company, had general charge of the work, assisted by Edward Hutchins, Assoc. M. Am. Soc. C. E., Assistant Engineer; Chester S. Colson, Hydraulic Engineer; W. H. Coventry, Electrical Engineer; H. M. Hale, M. Am. Soc. C. E., Resident Engineer; and A. H. Davison, Engineer Observer and Recorder. The writer was Consulting Engineer, and was assisted by Mr. Vincent J. McKinnon and George Perrine, M. Am. Soc. C. E. The work was carried out under a modified cost-plus contract and detail drawings were furnished by the Parklap Construction Corporation, of which Walter J. Douglas, M. Am. Soc. C. E., was President; Eugene E. Halmos, M. Am. Soc. C. E., Chief Engineer; S. A. Thoresen, Designing Engineer; E. A. Little, Resident Manager; G. Miller, M. Am. Soc. C. E., Field Engineer, and F. W. Barnes, Assoc. M. Am. Soc. C. E., Superintendent. J. P. Hogan, M. Am. Soc. C. E., gave advice at various times. The entire personnel of the International Paper Company and of the Parklap Construction Corporation deserves credit for faithful application under the many difficult conditions that arose during the progress of the work.

DEVELOPMENT OF HIGHWAY TRAFFIC IN CALIFORNIA*

By L. I. Hewes,† M. Am. Soc. C. E.

Synopsis

This paper compares for 1920 and 1922 the highway traffic count of the California State Highway System and adjoined county roads as taken by the U. S. Bureau of Public Roads and the State Highway Commission. For these two State-wide traffic counts there were 103 stations identical, covering 2 150 miles of the main State highway. The total average of all traffic per day for 1920 was found to be 1 382 vehicles and for 1922, 2 037 vehicles, an increase of 47.4 per cent. During this interval the motor-vehicle registration increased 54 per cent. In the 1922-23 count, 138 additional counting stations on State highways and 190 stations on adjoined county highways, making a total of 431 stations, were used. The paper gives an analysis of the character and volume of traffic at all these stations. All counts are based on an "equivalent" 16-hour day from 6:00 A. M. to 10:00 P. M. This day is built up by combinations of 8-hour fractions.

In California, truck traffic in 1920 was 125% of the total number and in 1922, 11.3 per cent. In the winter of 1922-23, truck traffic was about 8% of the total, corresponding with an average of about 10% for the whole year 1922-23. On the adjoined county roads, truck traffic was found to be about 11.5% of the yearly total.

By the use of "traffic profiles", which present the typical daily traffic for all the State highway routes, the total annual movement of all traffic on California State highways has been computed for 1920 and 1922. For the latter year, it was about 1 500 000 000 vehicle-miles.

A study was made of the characteristics of all truck traffic and some interesting figures are presented with respect to commodities hauled, overloads, length of haul, speed of operation, etc.

The number of persons per motor vehicle in California for 1923 was found to be 3.47, with an automobile registration of 1 100 283.

The supply of California motor vehicles is compared with the discharge or junking. From the curves of supply and discharge, it was calculated that the average life for the period beginning in 1915 was about 8 years.

The necessity of making better rates for gasoline and registration taxation is especially emphasized. The relationship between the public operating revenue on the highways and the necessary operating expenses for such highways

* Presented at the Meeting of the Highway Division, at Pasadena, Calif., June 19, 1924.

† Deputy Chf. Engr., U. S. Bureau of Public Roads, San Francisco, Calif.

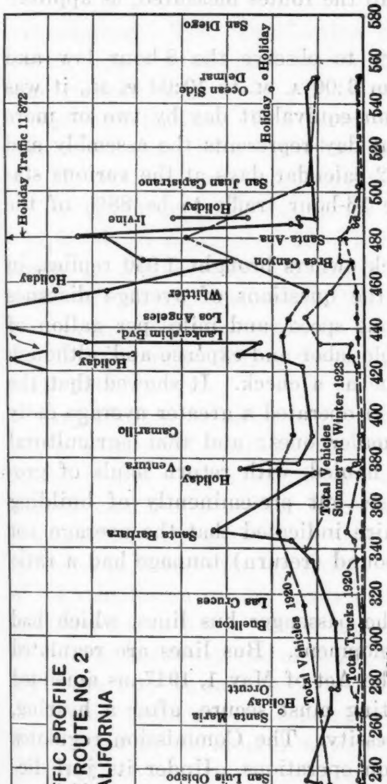
is found by computing the total annual traffic on two main State routes totaling 857 miles, between Los Angeles and San Diego, in the south, and Sacramento and San Francisco, respectively. At the current taxation, per vehicle-mile, the actual traffic on the selected routes does not meet the operating requirements of the highway, which are found to be \$2 428 per mile for maintenance, reconstruction, and improvement. The indicated operating income per vehicle-mile for 1923, based on the total traffic and the total gas and registration revenue, was about \$0.0018. At this rate there was a deficit of \$766 per mile based on the actual traffic and actual expenditures on the 857 miles observed. On the other hand, the actual returns to the State on the gas and registration revenue indicate a rate of about \$0.005 per vehicle-mile for the actual operation when prorated to the State highways only.

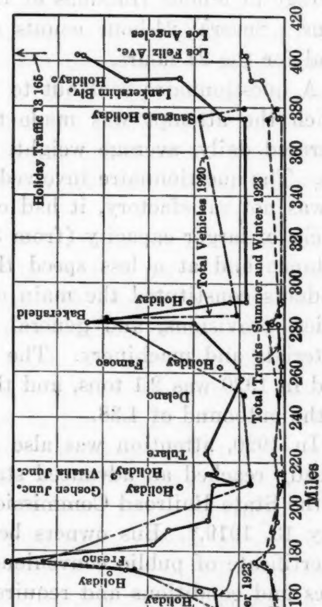
The position of California with respect to highway transportation is unique. Its large motor-vehicle registration, outdoor climate, and well developed highway system combine to make traffic exceptional. The measurement of this traffic and its bearing on other highway statistics is of general interest and value to highway engineers and highway economists. An indefinite number of statistical relations from the State-wide traffic counts might be set up, but it is the object of this paper to present only the major features of the traffic development and their economic significance.

Traffic Count of 1920.—Before 1920 a State highway traffic count had never been taken. That year, in connection with a study of the California highway system by the United States Bureau of Public Roads, a preliminary traffic count was made, on the basis of one equivalent 16-hour day, at 103 stations, on 2 150 miles of the main State highways. The count was intended to sample the summer traffic (June 1 to November 1), only. It occurred in the interval, August 7 to October 14, and gave a daily average of 1 382 vehicles per station between 6:00 A. M. and 10:00 P. M. Of this number, 12.5% was motor trucks and 2.3% horse-drawn vehicles.

There had been certain prior traffic counts made by Kern and Los Angeles Counties on selected highways, beginning about 1915, which showed a progressive increase in average daily truck traffic between 1915 and 1920 of 25 to 140 in Kern County, and of 95 to 330 in Los Angeles County.

The census of 1920 was intended principally to determine the usefulness of the California State highways and to measure the relation between traffic and existing conditions on 2 150 miles of roads. There were introduced, for conveniently determining the total traffic, traffic diagrams, or traffic profiles, on which the State highway routes were shown horizontally, and the daily traffic of motor trucks and vehicles vertically, both to suitable scales. (See Figs. 1 and 2.) The area below the traffic curves, in proper units, thus indicated the total daily traffic in vehicle-miles. From these diagrams for the several routes, the total daily use of the State highways was developed. Thus, the daily summer traffic in 1920 was estimated as approximately 2 500 000 vehicle-miles for the main surfaced highways, and the total for the summer


TRAFFIC PROFILE
STATE ROUTE NO. 2
CALIFORNIA
Total Vehicles
Miles
Total Vehicles 1920
Total Vehicles Summer and Winter 1923
Total Trucks 1920
Total Trucks - Summer and Winter 1923
Holiday Traffic 10 488
Holiday Traffic 13 892
Holiday Traffic 11 872
San Francisco
Colma
San Bruno
San Mateo
Palo Alto
San Jose
Gilroy
San Juan
Salinas
Gonzales
Soledad
San Lucas
Paso Robles
Atascadero
San Luis Obispo
Santa Maria
Orcutt
Buellton
Las Cruces
Santa Barbara
Ventura
Camarillo
Lankershim Bvd.
Los Angeles
Whittier
Brea Canyon
Santa Ana
Irvine
San Juan Capistrano
Ocean Side
Delmar
San Diego
Holiday
FIG. 1


TRAFFIC PROFILE
STATE ROUTE NO. 4
CALIFORNIA
Total Vehicles
Miles
Total Vehicles 1920
Total Vehicles - Summer and Winter 1923
Total Trucks 1920
Total Trucks - Summer and Winter 1923
Holiday Traffic 13 165
Sacramento
Galt
Lodi
Stockton
French Camp
Manteca
Salida
Modesto
Turlock
Merced
Califa
Madera
Fresno
Goshen Junc.
Visalia Junc.
Tulare
Delano
Famoso
Bakersfield
Saugus
Lankershim Blv.
Los Feliz Ave.
Los Angeles
Holiday
FIG. 2

interval alone, June 1 to November 1, for all the routes measured, as approximately 375 000 000 vehicle-miles.

In making this count it was necessary to observe the 8-hour law and instead of one continuous 16-hour day from 6:00 A. M. to 10:00 P. M., it was frequently found convenient to build up an equivalent day by two or more shifts. Hence, the count for the equivalent day represents the assembly and average of 8-hour fractions of from 2 to 12 calendar days at the various stations. Several 24-hour counts showed the 16-hour traffic to be 88% of the total for the 24 hours.

A questionnaire sent out to 21 000 truck owners brought 1 930 replies, in which the attempt was made to answer the questions of average distance operated daily, average weight, advantageous speed, and miles per gallon of gas. The questionnaire involved considerable labor and expense and, although it was not satisfactory, it had certain value as a check. It showed that the trucks of larger capacity (from 3½ to 7 tons) operated a greater average daily distance and at a less speed than the smaller ones; and that agricultural products constituted the main commodity hauled, with return hauls of groceries, provisions, and general merchandise, but pre-eminently of building materials and machinery. The questionnaire indicated that the average net load in 1920 was 2.1 tons, and that the inbound (return) tonnage had a ratio to the outbound of 1.38.

In 1920, attention was also given to the passenger bus lines, which had already reached an advanced stage of development. Bus lines are regulated by the State Railroad Commission, under the Act of May 1, 1917, as amended May 13, 1919.* Bus owners before operating must secure, after a hearing, a certificate of public convenience and necessity. The Commission regulates rates and conditions and requires reports on operations. Under its jurisdiction and protection against unfair and unnecessary competition and enforcement of schedules, etc., the passenger bus business has developed until, in 1920, there were 103 independent bus lines, excluding those with insufficient data. In the operation of these bus lines, were involved 3 075 miles of State highways and an average daily movement of 65 220 vehicle-miles, of which 46 046 were on State highways only. The longest trip then possible by purchase of through transportation tickets over connecting lines was from El Centro, *via* San Diego, Los Angeles, and San Francisco, to Portland, Ore., a distance of 1 448 miles. The capacities of passenger busses of the larger companies ranged from 8 to 22.

Traffic Count of 1923.—At the request of the California State Highway Commission, the U. S. Bureau of Public Roads co-operated in a second State-wide traffic census in 1922,† the total cost of which was $50 000.

For comparison, it was essential to count traffic at the same 103 stations that were occupied in 1920. In addition, 138 stations were established, which completed the original count and extended its scope from 2 150 miles to 3 200 miles, this including graveled sections and some graded mileage in the State system. A simple form of traffic-record blank, comparable to that in 1920,

* This law exempted bus lines operating prior to May 1, 1917.

† This census was actually completed in 1923.

was adopted. The classification of traffic was between passenger cars and trucks, again subdivided into light and heavy. Loaded and empty trucks were recorded separately. There was also provision for horse-drawn vehicles, busses, and trailers. Full instructions were issued giving the limits of the classes and details for filling in the headings and for tallying at cross-roads, etc. It was not found necessary to record the direction of traffic in 1922.

The summer traffic count for 1922 was commenced on August 27, and completed on September 20. This was followed by a winter count which began on January 25 and ended on March 3, 1923. The average summer traffic for all the stations in 1922 was found to be 2 037 total vehicles per day, an increase of 47.4% over the corresponding figure of 1920. For 1922-23 the daily average, during the winter alone, was 1 578, and during the whole year, 1 800.

In addition, 24 co-operating counties at 190 stations made traffic counts which, in general, included records of from 6 to 24 days per station, usually for 16 hours. The total number of stations at which traffic was counted in 1922-23 was, therefore, 431. On 135 of the county stations the counts synchronized sufficiently with the State counts to enable the resulting data to be included in State-wide statistics. The total daily arithmetic summer average of all vehicles on county roads was 722, except in Los Angeles County, where it amounted to 5 338. Since the county stations represent about 2 100 miles of county highways (estimated), adjoining the State highways, such county highways may be said to carry about 26% of the total traffic recorded.

Traffic profiles were constructed for the 1922 State census as shown on Figs. 1 and 2; the annual movement of motor-vehicle traffic on all State roads observed (about 3 200 miles) totaled 1 370 000 000 vehicle-miles. The daily average traffic on the same 2 150 miles counted in 1920 was, in 1922, approximately 2 750 000 vehicle-miles, an increase of about 13 per cent.* The 1922 count also represents an equivalent 16-hour day from 6:00 A. M. to 10:00 P. M. This 16-hour day, as in 1920, was built up by combinations and averages of about three thousand 8-hour fractions. For the most part, however, the 16-hour days in 1922-23 were continuous and both the summer and winter average daily counts are representative of an average of approximately four days each.

During the 1922-23 traffic count, trucks were weighed at eighty stations where the traffic was sufficiently heavy to justify the expense. Approximately 13 000 trucks and 1 100 trailers were weighed. It was found that, numerically, winter trucking was to summer trucking about as 55 to 100, and that truck traffic was approximately 10% of all traffic. Trucks classed by inspection as heavy (of more than 2½ tons capacity) were about 38.3% of the total number; the weighing statistics indicated, however, that 45.1% were heavy. About 55% of the total trucks were found to be loaded, in counting, and about 68% in the weighing; when traffic was very dense empty trucks were not weighed. Nearly 60% of the loaded trucks were overloaded with reference to their rated capacities. Of the trucks weighed, 11.7% were overloaded with respect to the 1921 law (700 lb. per lin. in. width of tires), and of these overloads,

* The apparent discrepancy in this percentage of increase and the 47.4% of increase in average traffic is due to the much greater accuracy of the 1922–23 traffic profiles.

82%, overloaded on the front axle, and 60%, overloaded on the rear axle, could have been avoided by re-distribution of the load.

The average net load in 1922 was 3.3 tons. Trucks of capacities of ½ ton to 1½ tons constituted 24.5%, of 2 to 2½ tons, 31.8%, of 3 to 4 tons, 27.2%, and of 5 to 7½ tons, 16.5% of the total number. The trucks of large capacity (5 to 7½ tons), however, carried 25.3% of the net loads weighed. Approximately one-half the truck traffic from a given point worked within a range of about 21 miles, although the average haul was 31 miles.

In 1922, 710 motor freight and passenger bus lines of all kinds were franchised under the control of the Railroad Commission. Some of the passenger lines carried express also, and some carried freight. Bus operators report annually to the Railroad Commission. Class A automotive transportation corporations having an annual operating revenue exceeding $20 000, numbered 61, of which 17 exceeded $100 000.* There were approximately 600 lines with incomes of less than $20 000. Many of the smaller lines have United States mail contracts, and carry passengers. The figures for 1922 show that the ratio of operating expense of the Class A lines to operating revenues, was 93.3 per cent. The total net operating revenue was in excess of $500 000. The average length of bus lines is apparently 41 miles. There appears to be a tendency toward consolidation, the most successful lines acquiring the smaller ones. On some of the highways bus traffic has become a considerable fraction of the total traffic; for example, between San Francisco and San Mateo, there are 200 busses daily; between San Mateo and San José about 115; between Los Angeles and Whittier more than 200; and north of Santa Ana about 130. The average rate per passenger-mile is apparently about 5.4 cents. Of 246 bus lines classified as to the length of route, 80, representing the largest class, operated from 25 to 50 miles; the total length of these lines is approximately 10 500 miles.

In filing applications for certificates of convenience and necessity with the Railroad Commission, freight trucking lines must show the proposed rates, schedules, and freight classifications. These lines usually make their own classification, but a gradual standardization is being evolved by the larger operators. The present rates vary from 20 cents per ton-mile for short hauls, to 8.3 cents per ton-mile for hauls of about 100 miles.

Comparison of 1920 and 1922 Traffic Counts.—In connection with the census of 1920, the U. S. Bureau of Public Roads attempted to ascertain certain characteristic or invariant elements connected with California highway transportation, such as the seasonal changes in traffic, the weekly periodic variation, the daily variation from hour to hour, the remainder of going and coming traffic, and the percentage of traffic between 6:00 A. M. and 10:00 P. M. Studies of speed were made for various classes of vehicles, with some useful results. All the 1920 results and deductions have since been refined and checked by the census of 1922.†

* Most of the lines having an operating income in excess of $100 000 were in operation prior to May 1, 1917.

† For details of the 1920 count see "Study of California Highway System", by U. S. Bureau of Public Roads, and for the 1922 count see "A Report of Traffic on State Highways and County Roads in California", 1922, both published by the California Highway Commission.

The seasonal variation (which was shown to exist in 1920 and merely approximated) was satisfactorily established in 1922, the ratio of total winter to total summer traffic being approximately 77%, except for trucks. Truck travel decreased much more than pleasure traffic during the winter, showing the importance of the harvest interval in California. The occurrence of only 58% of the weekly traffic during the first five days is characteristic. The remainder is on Saturday and Sunday. The daily variation of traffic is found to be characterized by a forenoon peak about 10 o'clock, and a greater afternoon peak at 5 o'clock. There is some deviation from this during the winter, when the afternoon peak seems to increase and the forenoon peak to decrease. The variation of total daily traffic differs somewhat from the truck traffic alone, for which the two peaks tend to be more nearly equalized and a greater percentage occurs prior to 6:00 A. M. The going and coming traffic so nearly equalizes (as it should) that, in 1922, no separate records of direction were made, and this simplified the counting. It was found in 1922-23 that about 92% of the total traffic occurred between 6:00 A. M. and 10:00 P. M., as compared with 88% in 1920. The higher percentage checks with the 1922 counts in several counties in Tennessee.

Speed tests on measured sections of the highway, made with a stop-watch in 1923, compared with the 1920 results without marked discrepancies. The 1922 tests were more accurate and show, on an average, a higher speed. Heavy passenger cars operate at high speeds, and heavy trucks at the lowest speeds, but commercial passenger busses operate at the highest speeds of all, approximately 33.1 miles per hour on an average. Table 1 shows the average week-day traffic at the comparable 103 stations for the two years, 1920 and 1922.

TABLE 1.—AVERAGE WEEK-DAY TRAFFIC AT 103 STATIONS.

	Number of Vehicles.					Percentage of Total Vehicles.				
	Summer.		Winter, 1922.	Average per day, 1922.	Counties.	Summer.		Winter, 1922.	Average per day, 1922.	Counties.
	1920.	1922.				1920.	1922.			
Total vehicles........	1 382	2 037	1 578	1 800	722	100.0	100.0	100.0	100.0	100.0
Total trucks only....	173	231	127	180	83	12.5	11.3	8.0	10.0	11.5
Passenger cars.......	1 146	1 724	1 388	1 547	604	83.0	84.7	88.0	85.9	83.7
Light trucks, loaded.	173	73	38	55	27	12.5	3.6	2.4	3.0	3.7
Light trucks, empty.		69	41	56	28		3.4	2.6	3.1	3.9
Heavy trucks, loaded		58	30	44	18		2.8	1.9	2.5	2.5
Heavy trucks, empty		31	18	25	10		1.5	1.1	1.4	1.4
Horse vehicles.......	32	24	17	21	17	2.3	1.2	1.1	1.2	2.3
Busses..............	31	35	31	33	6	2.2	1.7	2.0	1.8	.8
Truck trailers........	....	23	15	19	12	....	1.1	0.9	1.1	1.7

On the 2 150 identical miles covered both in 1920 and 1922, motor traffic was found to constitute 98.8% of the whole, as against 97.7% in 1920. The increase in total daily traffic was 47.4%, or from a summer arithmetic average of 1 382 vehicles in 1920 to 2 037 in 1922. In the corresponding interval, the

automobile registration had increased 54 per cent. On the county roads, also, 97.7% of all traffic in 1922 was found to be motor-driven.

In 1922, special attention was given to motor trucks of which 13 000 were weighed at 80 stations. The ratio of winter to summer trucks counted was as 55 to 100.*

By comparison with the results of the thorough traffic census taken in Connecticut in 1921 and 1922 by the U. S. Bureau of Public Roads, it is found that the truck traffic in California in general shows characteristics similar to that in Connecticut. In both States manufactured products predominate as the commodity hauled, and the percentage of empty trucks decreases with the increased length of haul. In California, light trucks (1½ to 2½ tons) were 56.3% of all the truck traffic and carried 40.1% of the total net tonnage; in Connecticut, similar light trucks were 66.6% of the total and carried 45.3% of all the commodities. There are, however, some interesting differences; for example, in California, there is a greater tendency for the loads to over-run the rated capacity of the truck; likewise the 1922 census showed that about one-half the trucks haul less than 21 miles, the average being approximately 31 miles, whereas, in Connecticut, the average truck haul was only 15.7 miles (for Connecticut trucks alone). In California, no particular correlation was observed between the width of the truck body and the overloading of the truck, although this correlation was observed in Connecticut.

Truck registration data in California in the past has not been satisfactory because, to 1924, commercial vehicles or trucks with pneumatic tires have been registered as automobiles and only vehicles with solid tires have been classed as trucks. This classification probably accounts for a decrease from 41 689 trucks registered in 1920 to 35 092 in 1921. Registrations for 1922 and 1923, however, show increases to 39 413 and 43 000 plus, respectively. In the 1920 census all trucks were counted as such irrespective of their tire equipment, and the daily average was found to be 173 at the 103 stations that were comparable with the 1922 count. In 1922, the truck traffic was found to be 231 exclusive of the pneumatic tired vehicles, which, in harmony with the registration procedure, were classed as heavy automobiles. It is thought that the small increase shown in truck registration from year to year is probably due to the adoption of pneumatic tires to an increasing extent for all lighter trucks, particularly commercial vehicles of the delivery type, and also to the relatively large increase in this class of commercial vehicles operating in cities. The motor-vehicle law as amended in 1923 now provides for separate registration of commercial vehicles, irrespective of their tire equipment; in 1924, to April, there had already been registered approximately 106 000 pneumatic tired commercial vehicles, either trucks or passenger vehicles, and 35 000 solid tired trucks. The total (141 000) immediately reflects the change in classification from the preceding year.

Discussion.—The progress of motor-vehicle registration in California since 1907 is shown in Table 2. This increase of registration compares with that for

* See "A Report of Traffic on State Highways and County Roads in California", published by the California Highway Commission.

the United States, as shown in Fig. 3, and with the increase of each of the five leading States, as shown in Fig. 4. Based on the 1920 census, with a conservative estimate of the increase of population, the number of persons per automobile in California in 1923 was approximately 3.47, and the average number of persons per automobile for the entire United States for 1923 was approximately 6.13. The ratio for California is not exceeded in any other State.

TABLE 2.—MOTOR-VEHICLE REGISTRATION IN CALIFORNIA BY YEARS.

Year.	Total Vehicles.		
	Number.	Increase.	
		Number.	Percentage.
1907*	10 020	....	....
1907	14 051	4 031	40.2
1908	19 561	5 510	39.2
1909	28 633	9 072	46.4
1910	44 122	15 489	54.1
1911	60 779	16 657	37.8
1912	91 194	30 415	50.0
1913	100 000	8 806	9.7
1914	123 504	23 504	23.5
1915	163 797	40 293	32.6
1916	232 440	68 643	41.9
1917	306 916	74 476	32.0
1918	407 761	100 845	32.9
1919	477 450	69 689	17.1
1920	583 623	106 173	22.2
1921	680 614	96 991	16.6
1922	861 807	181 193	26.6
1923	1 100 283	238 476	27.7

* Prior to April 8, 1907.

It is impossible to calculate accurately the total annual operation of motor vehicles in California. An estimate of the total annual movement on all State and county highways from the 1922-23 traffic census, is 3 000 000 000 vehicle-miles. However, the indicated probable annual vehicular movement, based on the consumption of gasoline for the last quarter of 1923, is about

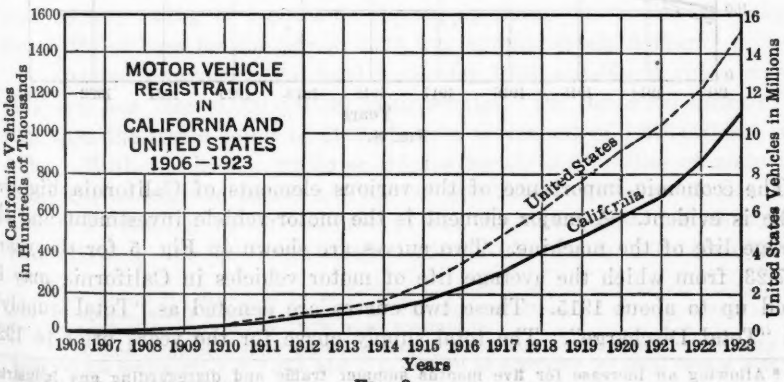


FIG. 3.

8 500 000 000 vehicle-miles; this mileage* was computed from the factors of (1) a gasoline revenue (2 cents per gal.) for the last quarter of 1923, of \$2.29 per car; (2) an estimated movement of 15 miles per gal.; and (3) a registration of 1 100 000 vehicles. At once, a very significant fact appears, namely, that a vast movement of vehicles occurs in the cities and towns. For rate-making purposes it will be desirable in the near future to have traffic counts within the cities of the State.

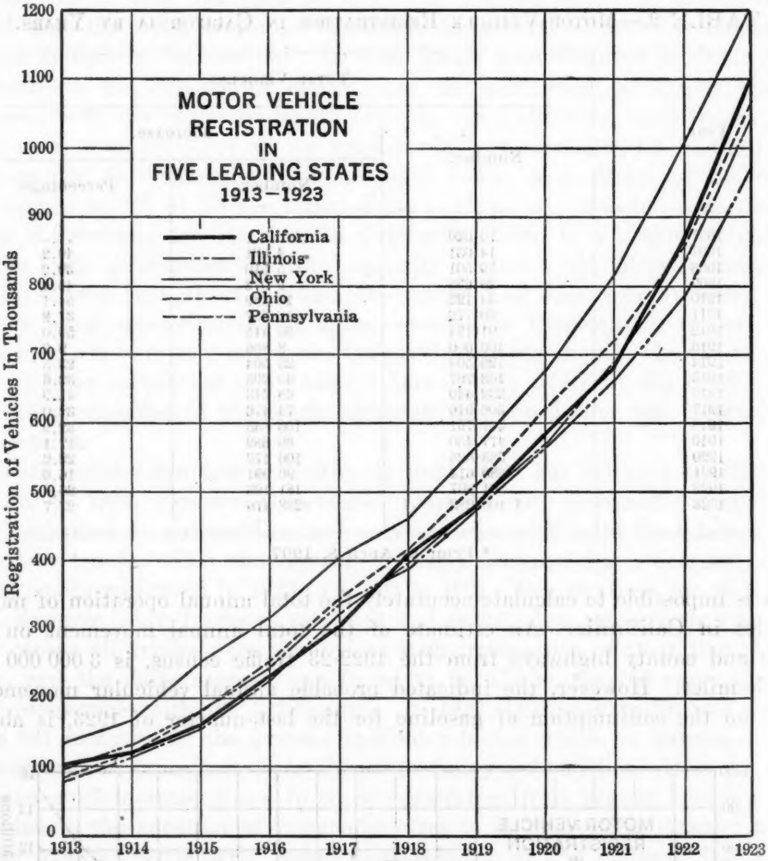


FIG. 4.

The economic importance of the various elements of California highway traffic is evident. A major element is the motor-vehicle investment and the average life of the machine. Two curves are shown in Fig. 5 for the years, 1907-23, from which the average life of motor vehicles in California may be found up to about 1915. These two curves are denoted as "Total Supply", and "Total Discharge". The total supply curve for the years prior to 1921

* Allowing an increase for five months summer traffic and disregarding any industrial use of gasoline because of constantly increasing registrations.

is obtained by calculation. From the total yearly production of the United States there is allocated to California an annual number of new cars and trucks, fixed by the ratio of registration of California to the total registration of the country for the corresponding year. From 1921-23, inclusive, the actual number of new cars sold has been taken. The total discharge curve is then found by subtracting from the total supply the California registrations

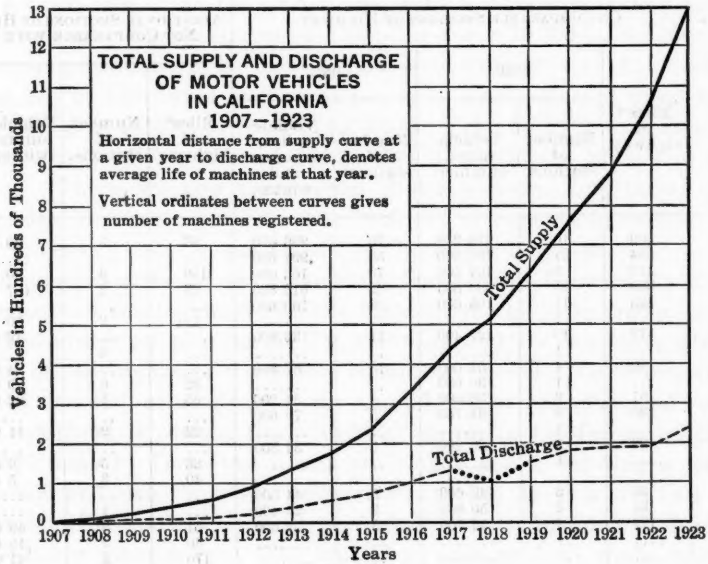


FIG. 5.

for corresponding years.* It is to be noted that both curves are cumulative; they represent the total machines supplied to or discharged from service at a given date. In Fig. 5 there are two interesting features: First, the vertical distance between the two curves represents approximately the registration for a given year; and, second, the horizontal distance between the supply curve and the discharge curve represents the approximate annual life of cars at the year corresponding to a point on the supply curve. It is seen that the indicated life of cars for the period 1915 was approximately 8 years.

As stated previously, the total traffic for 1923, outside of cities, was probably not less than 3 000 000 000 vehicle-miles. On the 3 200 miles of State highways the traffic count of 1923 shows a movement of 1 370 000 000 vehicle-miles. With the known traffic on approximately 2 100 miles of main county highways added, the figure is 1 850 000 000 miles, exclusive of Los Angeles County.† Adding as an estimate of Los Angeles traffic, 900 000 000 miles, and allowing for all extra traffic, 250 000 000 yields 3 000 000 000. With

* The dip of the discharge curve in 1918 is possibly due to demand for new cars exceeding production, which demand resulted in the return of discarded cars to service; or it may be that production figures for 1918 were incomplete.

† The Los Angeles County traffic is largely municipal in character. The daily average total vehicle movement was 5 338, and it is difficult to segregate the total county highway and State highway movement.

allowances for holiday traffic and unimproved sections of the total legal State system of 6 400 miles, the annual traffic on State highways alone for 1922-23 was probably not far from 1 500 000 000 vehicle-miles. Details of the State highway traffic studied by State routes are shown in Table 3.

TABLE 3.—AVERAGE VEHICLE-MILES PER DAY ON STATE ROUTES.

Route.	On Comparable Sections of Highway.					Additional Sections of Highway Not Comparable with 1920.		
	Miles* of highway.	1920.		1922.		Miles* of highway.	Number of stations.	Vehicle-miles, summer and winter, 1922.
		Number of stations.	Vehicle-miles, summer.	Number of stations.	Vehicle-miles, summer and winter.			
1	300	10	218 200	20	206 000	96	5	20 000
2	554	21	787 300	50	994 600	...	..	......
3	142	8	164 500	10	163 000	158	3	60 800
4	380	15	603 800	32	612 800	23	2	(67 000)†
5	125	11	165 600	23	166 600	...	..	......
6	....	1	....	..	....	15	2	30 400
7	176	10	127 400	11	132 800	...	..	12 000
8	....	1	....	..	....	...	2	......
9	70	4	57 000	10	87 600	...	..	24 600
10	....	1	36 000	..	....	32	6	29 800
11	51	3	36 000	4	31 200	65	1	22 000
12	130	3	64 100	5	79 600	...	..	......
13	....	1	....	..	....	22	2	11 000
14	33	3	56 200	5	34 800	...	..	......
17	....	1	....	..	....	26	5	6 000
18	....	..	....	..	....	40	3	5 400
19	30	2	49 600	5	44 600	...	..	......
23	36	2	50 900	2	26 400	...	1	......
26	122	2	56 700	5	74 600	51	3	49 000
27	....	..	....	..	....	40	3	15 600
28	....	..	....	..	....	176	2	41 000
29	....	..	....	..	....	102	2	19 800
31	....	..	....	..	....	80	2	23 800
37	....	..	....	..	....	115	2	34 800
Totals......	2 149	101§	2 437 300	182	2 654 600	1 041	46‡	406 000

* Refers to miles of State highway covered in census, but does not necessarily equal total miles of highway composing the various routes.

† Los Angeles County road, not included in total.

‡ There were thirteen stations on routes which were not worked up as profiles.

§ Four stations not shown on profiles, or combined with other stations in 1922.

Relation of Highway Traffic to Motor-Vehicle Tax Rates.—A study of highway transportation will be necessary in connection with the taxation of traffic. There is a growing conviction that highway traffic must be sufficiently taxed to pay the cost to the public for maintenance and upkeep and perhaps for at least a part of the construction. Taxation of such form and for such purpose is essentially rate-making for highway transportation and should follow closely the principles of other rate-making for public utilities. The two principal forms of motor-vehicle taxation now established—the motor-vehicle license fee and the gasoline tax—have exhibited in the past rapid changes and readjustments. In the future, rate-making for highway traffic must have a more scientific basis than in the past; the present rates are only approximations based on intelligent guesses.

With respect to the highway traffic income, the present is a period of rapid transition. Since 1905 the motor-vehicle license fees have progressively increased, and recently truck rates have been graduated on a capacity or weight basis. Rates have been established for passenger-carrying vehicles by classes, by weights, by capacity, and by horse-power. The present California law makes a basic rate of $3 for all vehicles, with a surcharge sliding scale for trucks by weight, the maximum of $40 being for a 10 000-lb. unladen truck. This surcharge appears to be about $25 for solid-tired trucks. At present, the gasoline tax in California is 2 cents per gal. The indicated gross revenue from gasoline in 1924 is at least $12 500 000. Prior to 1924 the motorists of California have paid a total of $34 939 338 to California State and county highways. There are now thirty-seven States with gasoline taxes, varying from 1 cent to 4 cents per gal. For 1923 the average motor-vehicle license fee for the entire country was $12.49; in California it was $9.64.*

The question arises as to the relation that obtains between the financial requirements of the highways and the taxes on the highway traffic. Clearly, the public operating revenue on the highways and the necessary operating expenses of the highways themselves should be related. Therefore, there is presented herewith an analysis of the annual cost of operation and revenues of two combined important State highway routes in California which total about 857 miles.† These two routes are Nos. 4 and 2, running south from Sacramento and San Francisco, respectively, to Los Angeles and San Diego, with practically continuous hard surfacing, mostly concrete and concrete with asphaltic concrete top. The profiles of average daily traffic as derived by the 1920 and 1922 traffic counts, are shown for these two routes, respectively, in Figs. 1 and 2. The costs of original construction on these routes are known, also the costs of reconstruction and improvement, and the approximate annual maintenance charge. From these the total annual cost per mile to the State for operating the combined routes is found to be approximately as follows:‡

Bond requirements, 6% on $19 270 construction costs (exclusive of Federal Aid)	$1 156
Maintenance and reconstruction	1 327
Improvement	1 101
Total	$3 584

* This figure does not include the gas tax effective October 1, 1923.

† Exclusive of mileage in towns and cities, which if included brings the gross mileage to about 975.

‡ Construction, taken from the State records, includes general items amounting to 14% of the distributed items, and, up to June 30, 1923, had amounted to $17 616 088, from which is subtracted $1 101 566 of Federal Aid funds. The remainder from bond revenues, $16 514 522, is equivalent to $19 270 per mile.

Maintenance and reconstruction, $1 327 per mile, is the sum of an average of $823 of general maintenance and $504, prorated reconstruction, for the period from June 30, 1922, to June 30, 1923, which compares with corresponding averages for the preceding three years of $656 general maintenance and $536 prorated reconstruction—total $1 192 per mile.

The prorated improvement figure of $1 101 per mile is the average per mile expended for the year June 30, 1922, to June 30, 1923.

Bond requirements of 6% include 1½% approximate amortization charges.

Of this total, the bond requirements are already met by general State taxes. There remains a balance of $2 428 to be derived from current revenue if this figure prevails for the succeeding years.

The actual portion of the total annual operating income from motor vehicles accruing to Routes 2 and 4 combined, totaling 857 miles, can first be determined by computing the revenue from the 2-cent gas tax and allotting a proper proportionate part* (9.22%) of the flat $3 fee and the surcharge license on trucks. From Table 2 the total vehicle-miles per day on the combined routes is 1 607 400. This figure must be increased for the night traffic and also for the week-end traffic. An increase of 8% for night traffic, and an addition of approximately 24% for week-end and special traffic, gives a total annual vehicle mileage for the two routes of approximately 784 000 000. There is then assumed an average movement per gallon of gas of 15 vehicle-miles with no allowance for the expense of collecting the gasoline tax. The average surcharge on the 43 000 trucks in the State in 1923 is assumed to be $25. Of these license fees 6% is chargeable to over-head. The following revenue results:

Total gas tax on 784 000 000 vehicle-miles at $0.001333...		$1 045 072
9.22% of $3 license fees on 1 100 000 vehicles....	$304 260	
9.22% of $25 surcharge fees on 43 000 solid-tired trucks	99 115	
	$403 375	
Less 6% for collection expenses...............	24 202	379 173
Total revenues, Routes 2 and 4...................		$1 424 245
Average per mile..................................		$1 662

This analysis of operating revenues and expenses is made from current data. Some adjustments, however, are required. As the truck registrations and total vehicle registrations for 1924 are not known, the 1924 rates are applied to the 1922-23 traffic figures. The total operating income per vehicle-mile on the 8 500 000 000 vehicle-miles annually, indicated for 1922-23, is $0.001847. This figure includes the prorated gasoline tax, $0.001333, the license fee, $0.000388, and the prorated truck surcharge, $0.000126. The indicated gross revenue is, therefore, about $15 700 000, which figure will doubtless be exceeded in 1924. Consequently, the apparent operating deficit of $766 per mile from the first analysis is subject to decrease unless the costs of maintenance, reconstruction, and improvement correspondingly increase with the traffic.

Another analysis may now be made. Since the State Highway Commission by law receives one-half the net motor revenues for maintenance and improvement of State highways, the current State highway income from this source is about $7 500 000. This income, divided by 1 500 000 000 vehicle-miles of indicated State highway operation, yields $0.005 per vehicle-mile of operating revenue. To the 784 000 000 vehicle-miles produced on Routes 2 and 4, there

* The total annual vehicle movement is taken at 8 500 000 000 vehicle-miles on all roads and streets. The movement on Routes 2 and 4 is 784 000 000 vehicle-miles, or 9.22 per cent. Truck traffic is assumed as 10% of all traffic, but the prorated truck surcharge is $0.000126 per vehicle-mile and the license fee, $0.000388 per vehicle-mile.

accrues then \$3 920 000 (\$4 574 per mile), or a surplus of \$2 146 over the current requirements on this route. If the bond requirements were included in the charges, the surplus would reduce to \$990 per mile. Routes 2 and 4 are the heaviest traveled long routes on the State highway system, and the operating revenue is greater in proportion to expense than the average of the entire system. A study of the entire system by the first method of analysis would reveal a larger proportionate deficit, and, similarly, by the second method probably a smaller surplus, than the figures here given for Routes 2 and 4.

It is interesting to examine the benefits from improved highways to the motor vehicles themselves. The cost of operating the motor vehicles in the United States has been variously analyzed, but the totals per mile do not differ greatly. Recent figures presented by F. R. White, M. Am. Soc. C. E., at a Congressional hearing show an indicated cost of about 10 cents per mile under average conditions for wearing surfaces of Portland cement concrete and asphalt or bituminous macadam.* This cost included depreciation, repairs, tires, oil and gasoline (stated at 85% of the total cost of operation), but not license fees, garage bills, insurance, etc., factors unaffected by the improvement of the highways. An average operating figure of about 10 cents per vehicle-mile may be checked by various computations and is probably representative of conditions in California for the average car. It is conservatively estimated by Mr. White that this figure represents a reduction of at least 2.5 cents over the comparable operating costs on unimproved highways.

The foregoing analysis necessarily contains some tentative assumptions, particularly with respect to the vehicle-miles per gallon of gasoline and the percentage of total annual operation of vehicles on the combined State highways and county roads. The analysis is given, however, to indicate a method by which highway operating costs and highway revenue may be compared. Manifestly, the difference between vehicle taxation and benefits to the vehicle by improved highways is still too wide to warrant over-refinement of the basic assumptions. Apparently, the benefit of the highway to the vehicle (\$0.025) is at least five times the cost of upkeep of the highway to the vehicle only (\$0.005), and this is probably the important element of modern highway construction.

There is perhaps no stronger reason for the continuation of State-wide traffic counts than the necessity for accurately measuring the annual traffic as a basis for determining the correct taxation of motor vehicles. The division of motor-vehicle revenues between State and local purposes requires further investigation. The principle, which was begun with the advent of State aid in Massachusetts and New Jersey in the early Nineties, of paying for country roads by city revenue, is being carried out in California in a new form under the present motor-vehicle taxation by reason of the evident preponderance of traffic in cities, from which the operating revenue accrues to the State and county highways. If the State highways are to be extended for the general benefit of the people of the State, this principle of taxing the city traffic for country roads must continue.

* *American Highways*, April, 1924.

FINAL REPORT OF THE SPECIAL COMMITTEE ON STRESSES IN STRUCTURAL STEEL*

Part I.—The Feasibility of Conducting Stress Measurements

TO THE BOARD OF DIRECTION,
AMERICAN SOCIETY OF CIVIL ENGINEERS:

Considerable research has already been done on the elements which form a steel structure, such as beams, columns, and tension members, and the results thus far have been of great value.

Within the past few years, an extensive investigation has been made of steel columns. The program was not carried out sufficiently to give more than a general report on a limited part of the subject; and a Research Committee of the Society has been appointed to continue the study. Rolled beams have also been tested, so that the engineers understand the general action of a steel beam, and, to a lesser degree, study has been made of the action of plate girders and riveted tension members.

When these members are put together in a structure with the complications of secondary and combined stresses, and when, in addition, in a building structure, the varying character of the dead and live loads are introduced, it will be seen that stress measurements in buildings involve a research problem in magnitude and time far above the average agency, and it does not seem feasible for the Society to attempt to carry on, unaided, such an extensive research program.

Stress measurements have been made on a number of steel frame buildings, but, in general, the work has been so limited in scope and so planned, that the conclusions are not susceptible of an analysis giving definite or valuable results. A recent report on stress measurements conducted by A. H. Fuller, M. Am. Soc. C. E., on a steel frame office building at Des Moines, Iowa, indicates that tests of this kind can be made of real value to engineers.

In its study of the subject, your Committee communicated with several commercial and engineering interests, and was able to arrange to have stress measurements made on several buildings, the work being somewhat in the nature of pilot tests, and from the results thus far obtained, your Committee is convinced that these and other interests will co-operate in future tests, if requested. Therefore, it is the opinion of the members of the Committee that if stress strain measurements were systematically conducted over a long period of time, much might be learned about the stresses carried by different members, and by the structure as a completed frame under varying conditions of loading; and it is felt that the work should be done, as the results will doubtless be of great value to the profession.

* Presented to the Annual Meeting, January 21, 1925.

Your Committee, therefore, recommends that a Research Committee be appointed to consider stress strain measurements, both in the individual members and in the assembled buildings. It is suggested that the work of the Committee is in no way dependent on the geographical representation and that if a small personnel be chosen from one district, the traveling expenses would be reduced to a minimum, probably $600 per year.

It is further suggested that the scope of work for the Committee and its duties be outlined as follows:

1.—That the work of the Committee should be one of supervision with limited field assistance, retaining for itself the digesting and analyzing of results; and that efforts should be made to suggest and encourage commercial and scientific agencies to make investigations, both on the elemental members and on the completed structure.

2.—That the Committee plan a comprehensive program and use its influence to have outside agencies co-operate in making the investigations necessary to carry out certain phases of this program, so that their work may be coordinated.

F. O. Dufour, *Chairman*,
R. J. Fogg, *Secretary*,
H. G. Balcom,
C. E. Chase,
O. F. Dalstrom,
J. H. Edwards,
Robert Farnham,
F. M. Masters,
L. D. Rights,
F. E. Schmitt,
W. J. Thomas,
L. J. Towne.

December 9, 1924.

Part II.—Working Stresses for Structural Steel in Buildings and Similar Structures

Majority Report

On August 9, 1922, the Committee was appointed by the Board of Direction to consider working stresses for structural steel in buildings and to consider also the feasibility of conducting stress measurements on steel structures. This report deals with working stresses. A separate report is submitted on stress measurements.

General Situation

Current working stresses for structural steel have been under active discussion for several years. Prior to the appointment of the Committee, a number of organizations and individuals had expressed themselves in favor of a revision of these stresses. In some quarters a basic working stress of 18 000 lb. per sq. in., in place of the generally used 16 000 lb., was advocated for the design of tension and bending members. The growing belief that existing stress practice was unduly conservative was brought to the attention of the Society by L. D. Rights, M. Am. Soc. C. E., in a letter addressed to the Board of Direction under date of July 21, 1921, in which he proposed that the Society undertake a study of working stresses with a view to recommending revision if such was found to be warranted. More recently various engineers have used an 18 000-lb. working stress in the design of important structures, and the trend in the direction of increased working stresses has found further expression in a recommendation in favor of the same value made to the principal trade association of steel fabricators (the American Institute of Steel Construction) by a commission of engineers employed by it to consider higher working stresses. A number of cities have incorporated the 18 000-lb. value in their building laws.

Steel building frame design in the United States has been based on a working stress of 16 000 lb. per sq. in., for bending and tension, during a large part of the past quarter century. Apparently, this value became established by empirical development beginning with the earliest days of steel frame building construction. At first, the designing practices and safety factors of the new art were based on bridge designing practice, itself at that time but partly evolved from rule-of-thumb method. It was usual to proportion by safety factors based on ultimate strength, following the custom developed in the application of rational designing to masonry and timber structures. Factors of 6 to 10, or more, were commonly applied to the earlier materials, and factors of 4 and 5, and sometimes 3, to the tougher wrought iron. A unit stress of 12 500 lb. per sq. in. came to be widely used for wrought iron, and when soft steel superseded this material allowable stresses were increased 25 to 30% to take advantage of its greater strength. By development of this kind, the stress of 16 000 lb. per sq. in. ultimately established itself in both bridge and building design. In the building field, it became fixed through

building-law requirements, although bridge design remained free from such control.

Characteristic practices as to allowed stresses in steel construction may be stated, as follows: Large bridges are virtually always proportioned for working stresses considerably above 16 000 lb.; unit stresses of 20 000 lb. are common. Many bridges of moderate span also have been designed to a basic stress of 20 000 lb. per sq. in., recommended by the late Theodore Cooper, M. Am. Soc. C. E., in his specifications. However, in recent years, the 16 000-lb. stress has come to be the most common in the design of new bridges of ordinary size, although bridges designed to the higher stresses of 20 to 25 years ago have proved entirely satisfactory in service. The change is admitted to represent an attempt to make provision for growth of live loads without increasing the nominal design loading beyond immediately prospective train and engine loads.

A different estimate of safety is applied to existing bridges that must be continued in service; such bridges are rated on the basis of working stresses of 24 000 to 26 000 lb. per sq. in. The latter figures indicate the confidence of engineers in the strength and permanence of steel when no evidence of weakness or structural deficiency is visible and when the loads are accurately known. Similar confidence finds expression in the use of high unit stresses, ranging up to 25 000 lb. per sq. in., in some important members of very large bridges, as suspension bridge towers, under a maximum combination of load and temperature.

In recent years, industrial buildings not subject to building-code restrictions have frequently been proportioned for working stresses of 18 000 to 22 000 lb. per sq. in. All such structures which have come under the personal observation of members of the Committee show entirely satisfactory behavior in service. Many older structures of the same type, built at a time when calculation of stresses was less precise and comprehensive than now, are of proportions corresponding to those that would be obtained to-day by the use of working stresses ranging up to 20 000 lb., or more. These structures also have shown no service weaknesses due to high stresses. Values higher than 16 000 lb. are generally used in the design of miscellaneous steel structures, such as towers and similar light framing. With these exceptions, American structural practice is quite generally limited to a nominal base stress of 16 000 lb. European practice is less conservative, using a stress of 17 000 lb. per sq. in. (12 kg. per sq. mm.) on the Continent, and 17 900 lb. (8 long tons per sq. in.) in Great Britain, generally with steels lower in strength and ductility than the material current in the United States.

In city building design certain arbitrary practices lead to lighter proportioning than would result from conscientious application of the prescribed building-code stress. They include ignoring the weight of columns and column covering when computing column loads, and making insufficient allowance for weight of movable partitions. In the case of high buildings, the first mentioned practice frequently places column design virtually on the basis of a 17 000 to 18 000-lb. short-column unit stress; the second practice is the equivalent of designing floor members for bending stresses much higher than

the nominal 16 000 lb. Such practices, the objectionable character of which is obvious, are believed to be due at least in part to a feeling that the 16 000-lb. base stress prescribed by the majority of building codes is unnecessarily low. In part, also, they may be attributed to survival of the old concept of a factor of safety expressed by the relation of ultimate strength of test specimens to calculated working stresses. The layman and the inexperienced designer still believe that there is a safety factor of 4, although engineers have long ago recognized that the margin of safety in fact is only 50 to 100 per cent. This safety-factor concept is believed to be thoroughly unsound and it is desired to express unreserved opposition to its retention.

Low working stresses are necessary in the early stages of the development of a material and an art. If, however, a fixed working stress persists from these early stages throughout a period during which engineering knowledge and skill steadily advance, the validity of this stress may properly be questioned. Careful review of past practice in steel building construction and experience with structures fails to reveal any justification for such persistence of basic working stress. Steady improvement in designing skill and in methods of construction has operated to increase the strength of members and details greatly. Not only has no use been made of this gain in the form of higher working stresses, but on the contrary a virtual decrease in working stresses has been brought about through the development of more precise and comprehensive calculation of stresses.

From an examination of service experience with structural steel work covering practically the past 30 years, the Committee feels warranted in stating that, although numerous instances of faulty performance can be found, no case of failure or unsatisfactory service is chargeable to high working stresses. All service deficiencies of early practice can be traced to causes unrelated to basic working stress, chiefly unskillful design of the general structural system, crude details, and insufficiency of bracing.

It is believed that early working stresses were conservative rather than the reverse, and that the record of service and the progress of the art are such as to warrant an increase in working stresses at the present time. The Committee is convinced that the demands of safety may be met by designing under higher working stresses, and, correspondingly, that continuance of the present stress basis is uneconomical and inefficient, and retards the development of an important branch of the constructive art.

Increased Working Stresses

In the attempt to arrive at a working stress better adapted to modern conditions, the Committee was not able to discover a method of rational calculation; methods of experimental and analytical approach to the problem fail, principally because building safety is more directly related to the chance of excess loading and other abnormal service than to variations of stress action within the structure. It has been found necessary therefore to base conclusions on the observed performance of structures during a long period of service, on the strength of materials under different conditions, and on prudent

estimation of prospective service requirements and contingencies in the light of past experience.

As permissible working stress is intended to define a maximum safe stress applicable to structures of widely varying character, subject to different conditions of service, degree of risk and future contingency, its amount is not susceptible of precise determination in any event. Working stresses, therefore, of necessity represent engineering judgment.

The Committee is strongly of the opinion that the proper application of working stresses to the design of engineering structures calls for engineering skill, and, further, that safe results presuppose reasonably capable and conscientious procedure both in design and in construction. Low intensities of stress may seem to give assurance against the harmful effects of lack of skill and improper work, but this assurance is dangerously fallacious. The statements and recommendations which follow are based on the premise that proper skill and care will be applied to designing, detailing, and construction.

Further, it is believed that safe building construction requires engineering knowledge and judgment to be so applied in determining the assumed loadings and their effects as to produce a structure the strength of which will be commensurate with the degree of hazard involved in its service. The fixing of a scale of working stresses for service of different degrees of hazard is not favored. The practical operation of building laws and municipal building departments at the present time, moreover, seems to require a single basic value of working stress.

Before reaching a conclusion as to what stress may be used safely in the general proportioning of steel building framework, the quality of commercial structural steel and the character of the service of such framework should be reviewed. The following paragraphs refer briefly to some of the factors involved.

Material

Commercial structural steel as made and used in the United States is a mild steel, well described by the specification of the American Society for Testing Materials (A9-24), which calls for an ultimate strength of 55 000 to 65 000 lb. per sq. in., a minimum yield point of 30 000 lb. per sq. in., but not less than half the ultimate, a minimum elongation of 1 400 000 divided by the tensile strength and a cold bend test flat for pieces not more than ¾ in. thick. Current tests show that a reasonably uniform quality is realized, so far as tensile properties are concerned. During the early stages of the manufacture of structural steel, there was much indefiniteness surrounding the quality of the commercial material. Several grades were in use, purchases were governed by numerous specifications, and the attempt on the part of the mills to meet the different demands resulted in unavoidable variation. As this variation was eliminated, and as the open-hearth process displaced the Bessemer process, a more dependable average quality of structural steel was obtained.

This report refers to structures built of steel meeting the structural steel specification of the American Society for Testing Materials, and does not apply to steel of unknown origin and qualities. Small quantities of steel

appear on the market and are found in the yards of fabricators as "stock material" lacking mill-test record or identification, but under present-day fabricating conditions such unidentified stock material is not an important element in steel construction. In so far as this class of steel is used in structures, it introduces an undesirable risk. The use of unknown material cannot be considered sound engineering practice, and it is not believed to be feasible to frame rules which would permit of safe use of such material under a reduced working stress. Where it is proposed to use steel the mill-test record of which is not available, the engineer should determine its properties by testing the several pieces to be used.

As most commercial structural steel is at the present time ordered under specification, subject to mill test, a representative lot of mill-test records of material of this grade, from six mills, widely scattered geographically and under various corporate ownerships, was collected with a view to obtaining a comprehensive check on its uniformity of yield point. The records were taken from about 3 500 heats of steel, and, as they were selected at fixed intervals from mill-test books covering a period of five or six years, they represent a very large tonnage. The heats were selected from those produced under the A. S. T. M. specification; they covered angles, beams, and channels, as well as shear and universal plates, of thicknesses ranging from ¼ in. to 1½ in.

The results of a study of the yield points of this material, reported to the Society in January, 1924,* showed a mean tensile specimen yield point of 36 500, a general range of yield point from 32 000 to 44 000, a moderate number of values (4.0%) exceeding 44 000, and a few (1.6%) lying below 32 000. The distribution of the yield-point values follows a smooth error curve, giving expression to the fact that the results embody the joint effect of variations in chemical composition, rolling temperature, cooling conditions, and testing-room procedure. Except for the presence of a certain quantity of steel averaging a yield point of about 43 000, erratic influences are noticeably absent.

Subsequent to the compilation noted, additional data were obtained from an analysis of a series of about 2 500 heat records supplied by a member of the Committee from his inspection books of steel purchased for a considerable number of buildings within the past few years. The average and range of this series were in close agreement with those cited. Taking both series together, the number of heats with yield points less than 32 000 was 1.1 per cent.

Although the average yield-point value of the tests lies between 36 000 and 37 000, the range of departure from this value is sufficiently wide to make it inadvisable to adopt the average as a basis for estimating the crippling strength of structural members. Since, further, present methods of commercial testing do not determine yield points accurately, conservative judgment suggests that working stresses be based on the low value of 30 000 lb, per sq. in. for yield point, even though the mill tests studied indicate that a higher value might be used.

It is known that there is much variation in the quality of steel as shown by tests of specimens cut from pieces of different sections, shapes, sizes, and

* *Proceedings*, Am. Soc. C. E., March, 1924, Society Affairs, p. 263.

thicknesses, rolled from ingots of the same chemical and physical characteristics. For material of moderate, uniform thicknesses, the ordinary methods of testing furnish a satisfactory index of the quality of the material going into the structure, but where thick material is used, the engineer should provide additional precautions in testing to assure himself that the material in the actual section used is still within the range of the material specification.

The necessary margin to be provided between load attack and resistance of structure was formerly estimated with reference to ultimate strength of material, but, in later years, has been referred to yield point. The Committee is unable to accept either view as wholly correct, and has considered working stresses with regard to both values. As heretofore stated, working stresses fixed as fractions of the ultimate strength of small test pieces by a so-called factor of safety do not give the reserve of strength indicated by this factor, although a strength margin above the yield point is effective under certain conditions of loading. It is the opinion that the margin of strength desired for normal service should be computed from the accepted yield-point value, that approximately defines the point at which crippling of the structure begins.

In the preceding the strength of steel has been expressed in terms of the strength developed in tension tests of small-sized test pieces, and engineers commonly refer to this same quantity. Working stresses, however, are more directly referable to the strength of full-sized members. The number of recorded tests of well designed, riveted, tension members is small, but these tests have shown yield points well above 30 000 lb., the limit indicated as a suitable strength datum. When it is considered that the yield point of a tension member does not represent its full strength, and that a considerable reserve is developed beyond this point, it appears conservative to base working stresses on the value of 30 000 lb. per sq. in. as the strength datum of full-sized members.

Attention is also directed to the fact that effective service strength is affected by the constant variations or oscillations of stress to which steel framework is subject in actual use. It has been shown by the most recent studies of the effect of repetitive loading, that the ultimate resistance of steel under varying stress, even when this variation is small, is approximately at the yield point of the material, sometimes being found to be slightly above this value. It may be concluded herefrom that the yield point properly represents a datum point to which all stresses other than those arising from extreme emergency conditions should be referred.

Service

As dealt with in this report working stresses are intended to be applied to the stresses induced by quiescent loads. The engineer should provide for excess stresses due to crowds, machinery, or other vibration-producing load by an impact addition to the static stresses, in order that the equivalent stresses so computed may be treated as static.

Wind pressures (and in the case of some types of structures other weather effects, such as snow and ice accumulation) represent an important uncertainty

affecting buildings. Experience shows, however, that the wind loads commonly used in design (in ordinary exposures, 25 to 30 lb. per sq. ft. of vertical exposed area) satisfactorily allow for even exceptional storm and tornado effects. For this reason only a moderate margin below the yield point is required when maximum wind and load are combined, and the reserve of strength which is depended on to resist extreme emergency loading and other contingencies need not be provided in addition to the wind-stress allowance, but may be estimated from the normal load condition, or the condition that is likely to be reached quite frequently in the life of a structure.

Structures are ordinarily proportioned to bear certain normal or reasonably foreseeable loads, and working stresses are applied to this condition. Insurance against abnormal and unforeseeable service must be supplied by an adequate margin between working stress and resistance of the structure to failure. It is not feasible to build strong enough to resist every conceivable happening, but a reasonable approximation to this ideal objective must be sought. Accordingly, the design should allow not only for uncertainties of the loading, variation of material, uncertainties of analysis and imperfections of construction, but also for emergency conditions. The latter include unforeseen forces, disturbance, violence, ordinary accident, or similar unusual happening. The relation of these abnormal to the normal requirements determines the insurance to be provided. The insurance should be ample, in view of the high degree of risk to human life which buildings usually involve.

Emergency effects cannot all occur simultaneously, nor are any two of them likely to attain their greatest value at the same time. Experience indicates that if the provision for emergency conditions is made sufficient to cover the one having the most severe effect, an adequate reserve of strength will be obtained, assuming that due allowance has previously been made for the factors of uncertainty in design, construction, and normal service.

Secondary stresses are known to occur in building frames, due to the distortion of the structure under its loads. Although these stresses have not been studied to any great extent, there is reason for believing that, in some cases, they may reach fairly large amounts. Their effect, however, is controlled by the predominantly quiescent character of the loading of buildings. Under such loading, secondary stresses are not believed to involve an important danger to the structure, since re-adjustment of a structure tends to neutralize them as the yield point is approached. No account, therefore, need be taken of secondary stresses in ordinary cases. In special cases, such as those involving the bending or twisting of columns at unbraced points, the engineer must make special provision for the stresses or neutralize their effect by proper bracing.

Consideration of all the factors of long-time service leads to the conclusion that, in the present state of the art, steel building frames and similar structures will be adequately safe if designed in tension, bending, and direct compression, to have a reserve of strength below the yield point of the structure equal to 50% of the total normal load effect. Past experience indicates that such a reserve is ample to cover load contingencies as well as imperfections of material and structure. The additional reserve between yield point and

ultimate strength of structure is available for chance conjunctions of unusual conditions and is also capable of furnishing resistance to momentary effects (such as shock) and to actions which are reduced by re-adjustment through yield.

The allowance suggested will ordinarily make the construction sufficiently rugged against wear and tear. In particular cases, as in very light structures, the designer may find it necessary to make separate provision to assure that his structure will be solid and substantial.

There has not been included in the margin between strength and working stress any "factor of ignorance" to cover unintelligent designing done by those not competent to determine the stresses to be provided for or unskilled in proportioning the parts of a load-carrying structure. Furthermore, it should not be considered that the recommended unit stresses have a safety margin for added loads or increased service beyond that known or contemplated. Any provision for such future change of service should be added to the stresses to be carried, at the time the design is made, and should not be considered as being provided for in the unit stress.

Tension

The tensile unit is considered the basic item in defining working stresses. From the value of 30 000 lb. per sq. in. as the yield point reasonably obtainable in the material, and a margin of 50% of the load stresses as a safe reserve under the conditions heretofore mentioned, the basic unit tension for quiescent load is determined as 20 000 lb. per sq. in.

Bending

Members designed to resist flexure in cross-bending are complicated parts of a structure. They must resist forces of tension, compression, and shear. The governing stress is usually the compression, and the part that resists compression may act in direct compression (when fully stayed laterally), or may act as a partly stayed column. Giving consideration to these conditions, a unit stress of 20 000 lb. per sq. in. in bending is recommended, when the compression flange is continuously rigidly held, and a reduced unit stress determined by the ratio of unsupported length to the width of the compression flange, when it is supported at intervals.

Column Stress

Columns have lower strength than members in tension or bending, and lack any considerable reserve of strength between yield and failure. Their working stresses, therefore, are controlled by separate considerations. Many facts about column strength remain to be investigated, especially the effect of different details and of structural constraints. The Special Committee on Steel Column Research of the Society is engaged in investigating such effects at the present time. However, there are sufficient test data on large columns to give reliable knowledge of integral column strength. In the light of these data, structural experience indicates that existing practice and building-law requirements lead to proportioning columns, especially those designed to carry heavy

loads, with a similar excess of material as tensile and bending members, and revision of their working stresses is believed to be equally necessary.

From the large number of tests of full-sized columns, up to sizes of about 100 sq. in. in cross-section, carried out in the last two decades, and particularly the extensive tests* made by the U. S. Bureau of Standards for the Special Committee on Steel Columns and Struts of the Society in 1914-17, it is known that well-built columns will bear, before failure, axial loads closely approaching the yield point of the material, or from 28 000 to 35 000 lb. per sq. in. They appear to be more sensitive to yield-point variations than tensile or bending members. Making allowance for such variations, however, a strength of 27 000 lb. per sq. in. may be expected as minimum from relatively short and thick columns (length ratio, $\frac{l}{r}$, not exceeding 50), and this only in the case of unusually soft material. Longer and more slender columns will have lower strengths. The amount of this decrease indicated by the U. S. Bureau of Standards tests, however, is less important for design than the desirability of providing adequate reserve strength against chance lateral forces in order to obtain substantial and durable structures.

It is believed that yield points determined in column tests are not of importance in estimating the service value of columns.

It is assumed that calculations of column loads will include proper allowance for eccentric application, as well as for bending moments due to wind and other definite lateral actions. No separate allowance is believed to be necessary for secondary stress and for such effects as may arise from settlement, unless unusual conditions exist, in which case the engineer's judgment must control the need and amount of special allowance. Secondary stress in columns affects mainly the end portions, where it does not materially influence the principal column action.

In view of the conditions affecting column strength in service, caution should be exercised in comparing testing-machine strength and service value of columns. The Committee holds that it is prudent to reduce somewhat the minimum testing-machine strength in estimating service value fairly, and to apply a rather larger reserve than the previously recommended amount of 50 per cent. Taking the service strength at 25 000 to 26 000 lb., and applying a 60% reserve, the permissible loading on relatively short columns is found to be 16 000 lb. per sq. in. It is thought that this is a safe figure, and it is recommended for columns up to a length-ratio of 50. Long and slender columns should be proportioned for much lower loads. Also, regardless of the yield-point strength of the material or testing-machine strength of short columns, it is recommended that a loading of 10 000 lb. per sq. in. for a length-ratio of 120 should not be much exceeded. The variation between the two stress limits indicated is preferably linear; and it is believed that the formula, 16 000 — 80 E, should be used for computing allowable column loading, the quantity, E, representing the excess of $\frac{l}{r}$ above 50.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–1920), p. 1583.

Summary and Recommended Stresses

Increased working stresses, as given in Table 1, may safely be used in the design of steel framework for buildings, provided certain conditions are met in the execution of the work, namely:

TABLE 1.—Recommended Unit Working Stresses* for Equivalent Static Stress in Structural-Steel Framework of Buildings and Similar Structures; to Be Applied to the Sum of Stresses for Load and Impact Addition; for Strength Margin of 50 Per Cent.

Kind of stress.	Permissible unit stress, in pounds per square inch.
Tension, net section	20 000
Compression, net section	20 000
Compression, axial, for columns with square ends or equivalent (with maximum of 16 000 up to $\frac{l}{r} = 50$)	16 000 − 80 E (E is excess of $\frac{l}{r}$ over 50)
Bending stress in extreme fibers of rolled shapes and built sections applied to net moment of inertia:	
When compression flange is continuously supported laterally.	20 000
When compression flange is not continuously supported laterally (with a maximum of 18 000 up to $\frac{l}{b} = 10$)	20 000 − 200 $\frac{l}{b}$ (l = unsupported length and b = width of flange)
Web shear, average on net section	15 000
Shear on power-driven rivets (including those driven with power operated hand hammer), pins, and finished bolts with driving fit.	15 000
Shear on hand-driven rivets and unfinished bolts	10 000
Bearing on power-driven rivets, pins, and finished bolts with driving fit:	
In single shear	27 000
In double shear	30 000
Bearing on hand-driven rivets and unfinished bolts:	
In single shear	18 000
In double shear	20 000

* In applying these stresses, proper attention must be given to web buckling and to deflection.

(1) That the material fulfills the requirements of the A. S. T. M. specification for structural steel for buildings, that the material has been properly identified by test, and that it has been determined that material of any heat, regardless of form of section or thickness of material, meets the minimum requirements of the specification.

(2) That the design is made by one competent to judge intelligently and correctly the loads to be carried and the service to be performed, to determine the resultant stresses, and to proportion the parts so that they will meet the conditions of well-recognized specifications governing the proportioning of parts and the fabrication and field erection of the structure.

(3) That the design is based on the actual dead loads and on the maximum live loads, increased by impact factors, consistent with the service of the structure (no future unknown change of service or loading is provided for in the units recommended); that the separate and combined stresses are correctly calculated; that the designing and detailing are properly done; and that unusual service conditions are separately taken into account by the engineer making the design.

(4) That first-class workmanship both in shop fabrication and in field erection is obtained, and that the engineer is in a position to assure himself that his design is satisfactorily carried to completion.

F. O. Dufour, *Chairman,*
R. J. Fogg, *Secretary,*
H. G. Balcom,
J. H. Edwards,
Robert Farnham,
L. D. Rights,
F. E. Schmitt,
W. J. Thomas.

December 9, 1924.

Minority Report

The undersigned members of the Committee are of the opinion that a moderate increase in working stresses for structural steel in buildings is justified on the basis of the present state of the art and existing information. They also believe it possible that a further advance to the basic stress of 20 000 lb. per sq. in. recommended by the Majority may be warranted after further study of the problem and after certain deficiencies in the present knowledge with regard to mill testing, shop practice, full-sized member strength, and methods of design are overcome. They do not believe, however, that the advance of 25% recommended by the Majority is warranted at the present time.

They believe that the conditions set forth in the Majority Report as a basis for the increase in stresses presupposes a type of supervision of design, testing, and construction which as a practical matter is secured in an exceedingly small portion of the total quantity of structural steel that goes into building work and that this fact must be taken into account in giving the approval of a committee of the Society to such a recommendation.

They do not consider that mill-test results for yield point, as recorded in commercial testing, afford a proper basis for determining the actual characteristics of the metal. They believe that when it is proposed to reduce the margin of safety in building construction it is highly important that there be no doubt as to the probable minimum strength of the structural members involved. Data presented to the Committee on tests of full-sized riveted tension members point to the probability that the range of yield point would be about 50% of the ultimate strength of the material, or about 27 500 lb. per sq. in. for steel with an ultimate strength of 55 000 lb. per sq. in. Short columns with metal not more than ¾ in. thick have a yield point not less than 25 000 lb. per sq. in. and an ultimate strength somewhat higher. Similar columns with metal 1 in. thick and more have failed at loads of about 25 000 lb. per sq. in., with yield points 1 000 to 3 000 lb. per sq. in. lower. The reduction in yield-point strength with the increase of thickness, of commercial structural steel, is well known.

Ordinary acceptance tests under standard specifications are not sensitive enough to detect the full extent of this reduction.

They believe that working stresses of 18 000 lb. per sq. in. for tension and bending and 15 000 lb. per sq. in. for short columns can safely be used for structural steel as at present specified and without limitation on thickness of material and with reasonably careful supervision of design and construction.

Before a proper decision could be reached on the proposal to adopt higher stresses than those cited in building work, they believe that a great deal more information on the subject is essential.

There should be a study of the material, covering at least the following points:

(*a*) Survey of requirements necessary to insure getting material of desired quality.

(*b*) Possibility of change in method of yield-point determination.

(*c*) Characteristics of thick material and precautions necessary in its use.

(*d*) Possibility of use of grades of steel of higher strength than the present standard. A stronger steel was in general use for building work twenty years ago than that used to-day. The minimum ultimate strength was formerly specified as 60 000 lb. per sq. in. This was changed to 55 000 lb. per sq. in. without reduction in working stresses.

Methods of fabrication should be investigated to determine whether there is need of any change in shop practices if higher stresses are to be used.

Tests would also be needed to settle certain questions relative to the strength of tension and bending members and to give information on the action of typical framed structures under load.

CLEMENT E. CHASE
O. F. DALSTROM
F. M. MASTERS
L. J. TOWNE

January 3, 1925.

SECONDARY STRESSES IN BRIDGES

Discussion*

By Messrs. Hardy Cross, W. M. Wilson, and Thomson E. Mao.

Hardy Cross,† M. Am. Soc. C. E. (by letter).‡—The author has done a very careful piece of work and has succeeded in stating with admirable clearness the standard methods for the computation of secondary stress. It is well that so comprehensive a statement is now available in American literature. The author's opinions as to the relative advantages of the different methods are of value, moreover, on account of his familiarity with all of them, for the writer has found that current statements as to the advantages of any one method are too often supported by complete ignorance as to all others.

In such computations, most engineers will combine from the standard methods those features which appeal to their individual tastes. The writer prefers algebraic methods of analysis; for structures with a large number of panels they are almost necessary as checks on graphic solutions. In using Mohr's method, therefore, he computes the rotation angles for the bars of the statically determinate truss. These may be easily found from the angle changes as computed by Manderla. It is to be noted that the true rotations are not required, but only relative rotations with reference to any bar, such as the center vertical, which is assumed to stand fast. If the changes of angle have been independently computed, this gives a complete check on the values of ψ.

The writer thinks the author does not give due prominence to the methods of successive approximation. For large trusses with many panels these seem decidedly the most practical of all, if not the only possible ones for office use. In this connection the author has omitted from his bibliography two interesting *Bulletins* of the University of Minnesota,§ which deal with successive approximations by Mohr's method. A method apparently almost identical with this has resulted from Swiss studies.‖ Although this method is said to be new, in its essentials it does not appear to be so. It is self-checking, converges with a reasonable number of approximations, and has a decided advantage in that the work can be picked up and carried on in spite of frequent interruptions. A method which requires uninterrupted attention is difficult to use in an office. The convergence has been sometimes misrepresented, the variations in successive values of ϕ at any point being shown, whereas it is successive values of $(\phi - \psi)$, which are really significant.

---

* Discussion on the paper by Cecil Vivian von Abo, Jun. Am. Soc. C. E., continued from February, 1925, *Proceedings*.

† Prof., Structural Eng., Univ. of Illinois, Urbana, Ill.

‡ Received by the Secretary, January 3, 1925.

§ *Bulletins 2* and *4*, Engineering Studies, Univ. of Minnesota.

‖ "Secondary Stresses in Steel Riveted Bridges", O. H. Ammann, M. Am. Soc. C. E., *Engineering News-Record*, October 23, 1924, p. 666.

Mao's analytical method is essentially the same as Müller-Breslau's, but complicated by the use of an unnecessary unwieldy and unfamiliar paraphernalia; the author is correct in discarding it. Müller-Breslau's method, however, is quite comparable, in simplicity and ease of application, with those of Manderla or Mohr. The modification of considering all clockwise moments as positive adds to its practicalness. The writer has used it for some years with modifications as explained herewith.

The author's Equations (47), (48), and (49)* may be rewritten, as follows:

$$\rho_4 = 2\,(\rho_1 - \rho_2 + \rho_3) - \rho_1 \pm (-\,6E\Delta\alpha) \qquad \text{(47a)}$$

$$\rho_5 = 2\,(\rho_1 - \rho_2 + \rho_3) - \rho_2 \pm (-\,6E\Delta\alpha + 6E\Delta\beta) \qquad \text{(48a)}$$

$$\rho_6 = 2\,(\rho_1 - \rho_2 + \rho_3) - \rho_3 \pm (+\,6E\Delta\beta) \qquad \text{(49a)}$$

The alternative signs before the angle changes will be taken as positive in those triangles in which the succession, ρ_1, ρ_2, ρ_3, etc., is clockwise, but reversed in those triangles in which the succession is anti-clockwise.

Now this re-arrangement makes possible a systematic computation of successive values in terms of ρ_1 and ρ_2. Moreover,

$$\rho_1 - \rho_2 + \rho_3 = \rho_4 - \rho_5 + \rho_6 \qquad \text{(119)}$$

which gives a repeated check on the additions.

Again, in a symmetrical truss, all the ρ's on the left may be expressed in terms of ρ_1 and ρ_2 and all the ρ's on the right in terms of ρ_{42} and ρ_{41},† the corresponding values of ρ at the other end of the truss. In corresponding ρ's, all but the constants will be identical on the two sides of the truss. The values of ρ_1, ρ_2, ρ_{42}, and ρ_{41} will then be determined by four equations at the center, still using the notation of Fig. 52 namely:

$$\rho_{21}\text{ left} = \rho_{21}\text{ right}$$

$$\rho_{22}\text{ left} = \rho_{22}\text{ right}$$

$$M_{20} + M_{21} + M_{23} = 0$$

$$M_{17} + M_{16} + M_{22} + M_{26} + M_{27} = 0$$

The first two will be found to be equations in $(\rho_1 - \rho_{42})$ and $(\rho_2 - \rho_{41})$ and the last two, equations in $(\rho_1 + \rho_{42})$ and $(\rho_2 + \rho_{41})$, thus furnishing two pairs of simultaneous equations which are easily solved.

When the values of the end ρ's have been determined, they may be substituted in the expressions already found, as the author himself has done.‡ It is, however, quicker to use again Equations (47a), (48a), and (49a), and carry these values through from each end to an absolute check by the four equations indicated at the center.

The writer finds convenient a form as indicated in Fig. 102 and Table 34. The values are taken in many cases from the author's computations; the checks are not always absolute, although the discrepancies are small.

Fig. 102 shows the position of the ρ's and thus the order in which they are taken up in Table 34. It will be noted that they are slightly different from the author's Fig. 52. ρ_{13} is repeated after ρ_{14}, as the computation here "back-

* *Proceedings*, Am. Soc. C. E., September, 1924, Papers and Discussions, p. 988.
† *Loc. cit.*, Fig. 52, p. 1046.
‡ *Loc. cit.*, p. 1049.

tracks" to get to ρ_{15}, and so around the triangle. The signs in Fig. 102 indicate whether the signs of the angle changes are to be taken as computed or are to be reversed.

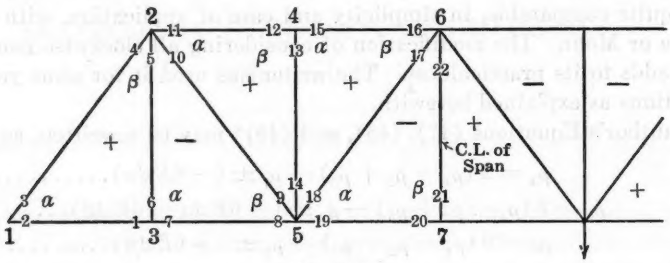


FIG. 102.

The procedure here indicated presents the following advantages:

1.—It systematizes both the writing and the solving of the equations in Table 8.*

2.—It adapts the work to the use of a calculating machine.

3.—It gives as the work proceeds an absolute check on all equations.

4.—It reduces the number of equations, and lessens the labor of their solution more than one-third.

5.—It simplifies the labor of evaluation shown in Table 9,† if the values found are "carried through" by the routine indicated. This also presents an absolute check on the work.

In sub-divided panels and similar cases it is necessary to introduce temporarily a third unknown which is evaluated by balanced moments in that panel. Four-sided figures such as occur in sub-divided trusses and in K-trusses may be solved by introducing a false member or by writing special equations. Müller-Breslau's method, however, is not so convenient in this very important group of cases as that of Mohr or Manderla.

Further simplification is also possible by rounding off the values of K and of the angle changes. In this particular case, the K-values for Members 2-4, 4-6, 6-8, 8-10, may well be taken as unity and the others in proportion, these ratios being rounded off to two places of decimals. Evidently, it is the K-ratios and not their absolute values which are significant in the equations. It is also evident that these ratios and the angular changes need not be computed with great precision provided exactly the same values are always used for the same terms. If this were not the case a slight difference from the computed value of the area or moment of inertia of a bar would upset the results to such an extent as to make all secondary stress computations futile.

This indicates another field of investigation, in which the writer is conducting some studies, not yet complete. Neither the length of member to be used in calculating the distortions of the truss, nor the length involved in the secondary distortion, is known with accuracy. In remains to be determined to what extent these uncertainties affect the values of the secondary

* *Proceedings*, Am. Soc. C. E., September, 1924, Papers and Discussions, pp. 1047–1048.
† *Loc. cit.*, p. 1049.

TABLE 34.—ANALYSIS OF SECONDARY STRESSES IN A 150-FT. TRUSS.

	K	ρ	COEFFICIENTS OF: ρ_1.	ρ_2.	CONSTANT: Left.	Right.	$6\,E\,\Delta$ ANGLE: Left.	Right.	FINAL VALUES: Left.	Right.
	2.501	1	+ 1	....	....	....	....	....	− 6.9907	− 15.3827
		2	....	+ 1	....	....	....	....	− 45.5976	+101.9963
		* 3	....	+ 0.4577	....	....	....	....	+ 20.8700	− 47.4233
	5.464	† Σ	+ 1	− 1.4577	....	....	....	....	+ 59.4769	−164.8023
		† 2 Σ	+ 2	− 2.9154	....	....	....	....	+118.9533	−329.6046
		4	+ 1	− 2.9154	− 96.66	+ 193.32	− 96.66	+ 193.32	+ 29.2845	−123.8582
	0.303	5	+ 2	− 3.9154	+ 39.90	− 79.80	+ 39.90	− 79.80	+204.4514	−511.4008
‡ Check		6	+ 2	− 2.4577	+ 136.56	− 273.12	+136.56	− 273.12		
		* 7	− 1.2424	+ 0.2979	− 16.55	+ 33.10	....	....		
	2.501	† Σ	− 1.2424	− 1.1598	− 113.21	+ 226.42	....	....		
		† 2 Σ	− 2.4848	− 2.3196	− 226.42	+ 452.84	....	....		
		8	− 4.4848	+ 1.5958	− 71.98	+ 143.84	+194.34	− 388.80		
	1.685	9	− 4.4848	+ 0.1381	− 6.22	+ 12.26	+356.76	− 713.70		
‡ Check		10	− 1.2424	− 2.6175	− 47.45	+ 94.84	+162.42	− 324.90		
		*11	− 0.5001	+ 2.7073	+ 74.93	− 149.91	....	....		
	7.951	† Σ	− 3.7425	+ 5.4629	+ 116.16	− 232.49	....	....		
		† 2 Σ	− 7.4850	+10.9258	+ 232.32	− 464.98	....	....		
		12	− 3.0002	+10.7877	+ 400.96	− 802.14	+162.42	− 324.90		
		13	− 6.2426	+13.5433	+ 800.27	−1601.06	+520.50	−1041.24		
‡ Check	0.190	14	− 6.9849	+ 8.2185	+ 515.47	−1031.41	+358.08	− 716.34		
		18	− 6.2426	+13.5433	+ 800.27	−1601.06	....	....		
		*15	+ 3.1494	−10.9841	− 420.09	+ 840.40	....	....		
	7.951	† Σ	+ 2.4071	−16.3089	− 704.89	+1410.05	....	....		
		† 2 Σ	+ 4.8142	−32.6178	−1409.78	+2820.10	....	....		
		16	+11.7991	−40.8363	−1771.59	+4275.35	+153.66	+ 423.84		
	1.800	17	+11.0568	−46.1611	−1908.73	+4992.66	+301.32	+ 571.50		
‡ Check		18	+ 1.6488	−21.6337	− 842.03	+2127.86	+147.66	+ 147.66		
		*19	+ 4.7670	+ 9.2408	+ 448.22	−1118.40	....	....		
	3.588	† Σ	+14.1750	−15.2866	− 618.48	+1746.90	....	....		
		† 2 Σ	+28.3500	−30.5732	−1236.96	+3493.80	....	....		
		20	+17.2932	+15.5879	+ 819.43	−1351.20	+147.66	+ 147.66		
	0.303	21	+26.7012	− 8.9395	+ 112.67	+1641.60	+507.60	+ 275.16		
‡ Check		22	+23.5830	−39.8140	−1325.24	+4739.70	+359.94	+ 127.50		

Continued to a check at the center line on all these values.

$$\rho_{21} = \rho_{21}' + 26.7012\,(\rho_1 - \rho_1') - 8.9395\,(\rho_2 - \rho_2') = +\,1528.93$$
$$\rho_{22} = \rho_{22}' + 23.5830\,(\rho_1 - \rho_1') - 39.8140\,(\rho_2 - \rho_2') = +\,6064.94$$

$$\rho_1 - \rho_1' = +\,8.3920; \quad \rho_2 - \rho_2' = -\,147.5939$$

$$M_{20} + \frac{1}{2}\,M_{21} + M_{20}' + \frac{1}{2}\,M_{21}' = 0 \qquad +\,18.4012\,(\rho_1 + \rho_1') + 15.2104\,(\rho_2 + \rho_2') = +\,457.27$$

$$M_{16} + M_{17} + \frac{1}{2}\,M_{22} + M_{16}' + M_{17}' + \frac{1}{2}\,M_{22}' = 0 - 14.7509\,(\rho_1 + \rho_1') - 52.0536\,(\rho_2 + \rho_2') = -\,3266.82$$

$$(\rho_1 + \rho_1') = -\,22.3734\,\rho_2 + \rho_2 = +\,56.3987$$

$$\rho_1 - \rho_1' = +\,8.3920 \qquad \rho_2 - \rho_2' = -\,147.5939$$
$$\rho_1 + \rho_1' = -\,22.3734 \qquad \rho_2 + \rho_2' = +\,56.3987$$

$$\rho_1 = -\,6.9907; \; \rho_1' = -\,15.3827; \; \rho_2 = -\,45.5976; \; \rho_2' = +\,101.9963$$

*These values of ρ are obtained from equations of balanced moments at the joints, these being the first five and last five equations indicated by the author (*Proceedings*, Am. Soc. C. E., September, 1924, p. 1047).

† These values are the sum of the first and last less the middle one of the three values just preceding.

‡ The word, "check", indicates that a check is here applied to the summation, as indicated in Equation (119). This check is absolute on the continued summation, but not on the solution by moments for the values of ρ. The latter may be checked by assuming any convenient values, such as 10 for ρ_1 and ρ_2, and then testing the equation, $\Sigma\, M = 0$.

stresses; it may be that only a roughly approximate solution is justifiable, in which case the method of successive approximations has even greater advantages. While, therefore, the mathematical theory of secondary stresses is quite complete and little more is to be expected from its elaboration, there is still room for systematizing the computations and especially for simplifying them by approximations and for determining what degree of approximation

is desirable or permissible. The writer believes that, in general, any relative advantages of the methods of Manderla, Mohr, and Müller-Breslau, when these methods are carried out in full, are largely illusory, but it does appear that Müller-Breslau's method is very readily systematized while the first two—which are practically identical—are more readily adapted to successive approximation.

Whatever is done, the computation of secondary stresses will remain a strenuous pursuit and at the end of the chase one faces the problem of what to do with the quarry. The cases will probably not be numerous in which forcible measures of elimination are justified. In other cases, evaluation gives some idea of possible points of weakness. High secondary stresses may then be reduced by revising the make-up of the members or by such devices as adjusting hangers and idle struts. The writer, however, believes that it is not practicable at present to effect a better distribution of metal by designing for the total stress (using generally increased basic stresses) rather than for the primary stress. The following uncertainties bear on the question:

(*a*) Secondary stresses in tension members are probably less serious than in compression members;

(*b*) Secondary stresses in compression members are less serious when there is double flexure (with a point of contraflexure in the member) than otherwise and they may even be beneficial in such cases in that they retard buckling;

(*c*) Secondary stresses are often less dangerous than primary stresses because of their localization, this being especially true when secondary moments in two planes are combined;

(*d*) Present knowledge of impact effects on secondary stresses is much less than in regard to primary impact, inadequate as are the data on the latter; perhaps secondary stresses tend to dampen vibrations and an unformulated knowledge of this supports the structural engineer's preference for "stiff" structures in spite of their secondary stresses.

These statements, however, should not be taken to indicate any doubt as to the growing importance of a thorough understanding by bridge engineers of the nature, approximate theoretical value, and general theory of secondary stresses in bridge trusses; but the same importance attaches to the study of secondary stresses due to the deformation in the frames of steel buildings and in other cases. There is, at present, a decided swing toward more elaborate analyses in both steel and concrete structures; the requirement in the new specifications for concrete of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete* that bending stresses shall be computed in the columns of concrete buildings, even in the face of uncertainties more pronounced than those outlined herein, is an instance in point. All this is very well as long as it is understood that such analyses merely indicate danger points. Elaborate analyses alone, however, will not justify a general increase in fiber stress; the relation of secondary stresses to the factor of safety is a much more intricate problem than their mathematical formularization or evaluation.

* *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions, pp. 1204–1206.

W. M. Wilson,* M. Am. Soc. C. E. (by letter).†—The Engineering Profession is indebted to the author for presenting, in a form for comparison, the various methods that have been developed for computing the secondary stresses in the members of a truss due to axial deformations. The selection of a method usually embodies an accidental element of considerable importance for, with most engineers, the first method that has been mastered and systematized seems better than a new one and is likely to be retained, even though the new one may be superior.

The writer prefers the method that the author has designated as Mohr's semi-graphical method. This is certainly the easiest to understand and, as indicated by the author, is as short as any. Moreover, each operation can be checked as it is performed, thus eliminating costly errors that might otherwise be undetected until revealed by equating the sum of the moments at a panel point to zero. Furthermore, the extensions can be made with sufficient accuracy by a 20-in. slide-rule. Nor will the writer concede that the graphical work (Williot diagram) introduces errors large enough to be objectionable; he has used the method extensively and has supervised its use on work where a high degree of accuracy was necessary, and has not found it wanting.

The writer recently introduced his graduate class in advanced stress analysis to the subject of secondary stresses. For the first problem he required the nine students to determine the secondary stresses at both ends of all members of a structure consisting of a single triangle, *ABC*. The moments computed for the ends of the members, are listed in Table 35. It is realized that the problem, as problems in secondary stresses go, is simple. Moreover, the results obtained by the various students do not agree very closely in some cases; but, considering that, in each case, the student was working his first problem in secondary stresses and using his own Williot diagram, the discrepancies are not large. Moreover, the students who obtained answers differing most from the average also failed to obtain summations of moments at the panel points equal to zero. Inasmuch as a failure to obtain equilibrium is due entirely to inaccuracies in the arithmetical work rather than in the graphical part of the solution, the latter cannot be charged with all the discrepancies in Table 35.

Recently, the writer was engaged by a firm of consulting engineers to compute secondary stresses. Knowing that he used the semi-graphical method, the firm, unknown to him, submitted the same structure for analysis to another engineer who uses an algebraic method; the firm redeemed itself for whatever breach of ethics it may have committed by reporting to the writer that the secondary stresses, as computed by the algebraic method, agreed almost exactly with those obtained by the writer using the Mohr semi-graphical method.

As another example cited to establish the accuracy of Mohr's semi-graphical method, the writer had the secondary stresses in a five-panel Pratt truss computed by two methods, as follows: Method 1, by determining the secondary stress at a certain point due to the primary stress in some one member, all other members being unstressed; likewise determining the secondary stress at the

* Research Prof., Structural Eng., Univ. of Illinois, Urbana, Ill.

† Received by the Secretary January 8, 1925.

same point due to the primary stress in each of the other members considered separately; and Method 2, by determining the secondary stress at the specified point when all members are stressed simultaneously. Barring inaccuracies in the work, the sum of the separate secondary stresses obtained by Method 1 will equal the secondary stresses obtained by Method 2.

TABLE 35.—SECONDARY MOMENTS IN TRIANGULAR FRAME.

Moment.	Student No.								
	1.	2.	3.	4.	5.	6.	7.	8.	9.
M_{AB}	−285 000	−290 000	−288 000	−288 000	−284 000	−300 000	−286 000	−286 200	−291 600
M_{AC}	+293 400	+291 000	+291 000	+291 600	+302 400	+284 000	+283 200	+288 300	+290 700
M_{BA}	−642 000	−651 000	−648 000	−648 000	−649 200	−654 000	−656 000	+645 800	−648 600
M_{BC}	+655 600	+651 000	+653 000	+648 000	+652 500	+655 200	+652 000	−646 200	+650 400
M_{CB}	+169 200	+175 000	+176 500	+172 800	+174 000	+190 800	+168 000	+174 400	+175 000
M_{CB}	−169 000	−175 000	−174 500	−172 800	−178 800	−174 600	−171 000	−174 600	−174 600

The secondary stresses were computed for thirty-six points, one at each end of all the eighteen members, with the following results: The difference between the secondary stresses obtained by the two methods in no case exceed 3% of the total; at two points, the difference exceeded 2% and was less than 3% of the total; at four points, the difference exceeded 1% and was less than 2% of the total; at thirty points, the difference was less than 1 per cent. The differences exceeding 2% occurred at points where the secondary stress was very small, being less than 1½% of the maximum secondary stress; the differences exceeding 1% occurred at points where the secondary stress is less than 20% of the maximum secondary stress. The degree of accuracy attained is thus much greater than is necessary. The writer's experience, not only in the cases cited, but in many others as well, has convinced him that the method is accurate enough for all practical purposes. The Williot diagrams, however, must be carefully drawn.

The cross-frame treated by the author* is a special case of the rectangular frame and may be solved by the equations that have been developed for rectangular frames in general.†

For the frame represented by Fig. 103, symmetrical about a vertical center line, but loaded unsymmetrically:‡

$$M_{AB} = -\frac{1}{2}\left\{ C_{HA}\left[\frac{2n+3p}{\alpha} - \frac{1}{\beta}\right] + C_{AB}\left[\frac{2n+3p}{\alpha} + \frac{1}{\beta}\right]\right\}$$

$$M_{BC} = -\frac{1}{2}\left\{ C_{BA}\left[\frac{2n+3p}{\alpha} + \frac{1}{\beta}\right] + C_{AB}\left[\frac{2n+3p}{\alpha} - \frac{1}{\beta}\right]\right\}$$

* *Proceedings*, Am. Soc. C. E., September, 1924, Papers and Discussions, pp. 1095-1111.

† See *Bulletin No. 108*, Eng. Experiment Station, Univ. of Illinois. "Analysis of Statically Indeterminate Structures by the Slope Deflection Method", by W. M. Wilson, F. E. Richart, and Comillo Weiss, p. 105.

‡ *Loc. cit.*, p. 107.

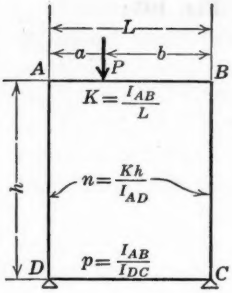


FIG. 103.

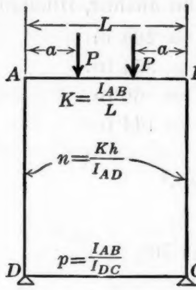


FIG. 104.

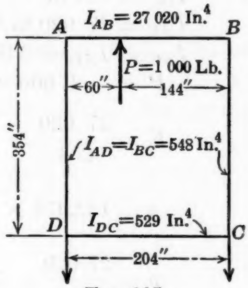


FIG. 105.

$$M_{CD} = \frac{1}{2} \left\{ C_{BA} \left[\frac{n}{\alpha} - \frac{1}{\beta} \right] + C_{AB} \left[\frac{n}{\alpha} + \frac{1}{\beta} \right] \right\}$$

$$M_{DA} = \frac{1}{2} \left\{ C_{BA} \left[\frac{n}{\alpha} + \frac{1}{\beta} \right] + C_{AB} \left[\frac{n}{\alpha} - \frac{1}{\beta} \right] \right\}$$

in which,

$\alpha = n^2 + 2\,p\,n + 2\,n + 3\,p$

$\beta = 6\,n + p + 1$

n = ratio of K (moment of inertia divided by length) of top member to K of left-hand column, for a rectangular frame.

p = ratio of K of top member to K of bottom member for a rectangular frame.

M_{AB} = moment at End A of Member A-B.

M_{BC} = moment at End B of Member B-C.

M_{CD} = moment at End C of Member C-D.

M_{DA} = moment at End D of Member D-A.

C_{BA} = moment at End B of Member A-B, due to any system of loads, when A and B are on the same level and when the tangents to the elastic curve at A and B are horizontal.*

C_{AB} = moment at End A of Member A-B.

For a frame symmetrical about a vertical center line and supporting loads symmetrical about the same center line, the condition usually found in practice, the moments are given by the equations:

$$M_{AB} = M_{BC} = -\,C_{AB}\,\frac{2\,n + 3\,p}{\alpha}$$

$$M_{CD} = M_{DA} = C_{AB}\,\frac{n}{\alpha}$$

In these equations, C_{AB} and C_{BA} are functions of the loads. For the single load shown in Fig. 103, $C_{AB} = \dfrac{P\,a\,b^2}{L^2}$† and $C_{BA} = \dfrac{P\,a^2\,b}{L^2}$. For the two symmetrically spaced equal loads of Fig. 104,

$$C_{AB} = C_{BA} = \frac{P\,a}{L}\,(L - a).$$

* Values of C for various loads are given in *Bulletin No. 108*, Eng. Experiment Station, Univ. of Illinois, pp. 24, 26.

† See *Bulletin No. 108*, Eng. Experiment Station, Univ. of Illinois, pp. 24, 26.

For the problem solved by the author, illustrated by Fig. 105:

$$I_{DC} = 529 \text{ in.}^4 \qquad L = 204 \text{ in.}$$
$$I_{AB} = 27\ 020 \text{ in.}^4 \qquad h = 354 \text{ in.}$$
$$I_{BC} = I_{AD} = 548 \text{ in.}^4 \qquad a = 60 \text{ in.}$$
$$P = -1\ 000 \text{ lb.} \qquad b = 144 \text{ in.}$$

$$K = \frac{27\ 020}{204} = 132.451 \text{ in.}^3$$

$$n = 132.451 \times \frac{354}{548} = 85.561$$

$$p = \frac{27\ 020}{529} = 51.078$$

$$C_{AB} = \frac{-1\ 000 \times 60 \times 144 \times 144}{204 \times 204} = -29\ 896 \text{ in-lb.}$$

$$C_{BA} = \frac{-1\ 000 \times 60 \times 60 \times 144}{204 \times 204} = -12\ 456 \text{ in-lb.}$$

$$\alpha = 85.561^2 + 2 \times 51.078 \times 85.561 + 2 \times 85.561 + 3 \times 51.078$$
$$= 16\ 380$$

$$\beta = 6 \times 85.561 + 51.078 + 1 = 565.444$$

$$\frac{2n + 3p}{\alpha} = \frac{2 \times 85.561 + 3 \times 51.078}{16\ 380} = 0.01980$$

$$\frac{1}{\beta} = \frac{1}{565.444} = 0.001769$$

$$\frac{n}{\alpha} = \frac{85.561}{16\ 380} = 0.005223$$

$$M_{AB} = -\frac{1}{2}\Big[-12\ 456\ (0.019802 - 0.001769)$$
$$-29\ 896\ (0.019802 + 0.001769)\Big] = 434.6 \text{ in-lb.}$$

$$M_{BC} = -\frac{1}{2}\Big[-12\ 456\ (0.019802 + 0.001769)$$
$$-29\ 896\ (0.019802 - 0.001769)\Big] = 403.8 \text{ in-lb.}$$

$$M_{CD} = \frac{1}{2}\Big[-12\ 456\ (0.005223 - 0.001769)$$
$$-29\ 896\ (0.005223 + 0.001769)\Big] = -126.0 \text{ in-lb.}$$

$$M_{DA} = \frac{1}{2}\Big[-12\ 456\ (0.005223 + 0.001769)$$
$$-29\ 896\ (0.005223 - 0.001769)\Big] = -95.2 \text{ in-lb.}$$

The writer made the extensions with a 20-in. slide-rule and the results are in almost exact agreement with those obtained by the author using Mohr's method.

For a cross-frame having two equal symmetrically spaced loads (the usual condition), the computations are very short and simple. Using the same frame as before, but considering two loads, each equal to 1 000 lb., the computations are as follows:

$$M_{AB} = M_{BC} = -C_{AB}\frac{2n + 3p}{\alpha}$$

$$M_{CD} = M_{DA} = C_{AB}\frac{n}{\alpha}$$

in which,

$$C_{AB} = \frac{Pa}{L}(L - a) = \frac{1\,000 \times 60}{204} \times 144 = -42\,350 \text{ in-lb.}$$

As previously determined:

$$\frac{2n + 3p}{\alpha} = 0.01980$$

$$\frac{n}{\alpha} = 0.005223$$

Substituting these quantities in the equations for the moments, gives:

$$M_{AB} = M_{BC} = 42\,350 \times 0.01980 = 838.3 \text{ in-lb.}$$

$$M_{CD} = M_{DA} = -42\,350 \times 0.005223 = -221.2 \text{ in-lb.}$$

These computations are so short and simple and the secondary stresses in the cross-frame are so important that the omission of the calculations for secondary stresses from the design seems hard to justify even in structures of moderate size.

The equations used by the writer and by the author are equivalent; the writer believes, however, that his form* is the more convenient for the designer.

THOMSON E. MAO,† ESQ. (by letter).‡—The writer desires to pay his highest respects to the author for the exhaustive treatment of the subject and the splendid presentation of the writer's graphic and analytic methods for the solution of secondary stresses. As far as the writer knows the paper is the most complete study of secondary stresses in the English language, and yet is not too mathematical for perusal by busy engineers in practice. The author is to be congratulated for this painstaking work, for unless a person has studied the subject deeply and actually solved some examples he cannot really appreciate the amount of time and patience required to prepare such a comprehensive paper. This work appeals particularly to the writer, who was, for some time, completely "immersed" in the subject of secondary stresses and has longed to find a companion who would have exactly the same taste and interest in such an unpopular and "lonely" subject. From the fact that secondary stresses are bound to be a factor in design, the author also deserves the appreciation of the

* Presented in *Bulletin No. 108*, Eng. Experiment Station, Univ. of Illinois.

† Pres., Conservancy Eng. Coll., Nanking, China.

‡ Received by the Secretary, January 9, 1925.

Engineering Profession at large, as he has again proved at great expense of time and labor that, after all, this important branch of stress computation could not well be taken care of by the "factor of ignorance."

This discussion will largely deal with the two methods introduced by the writer in 1919 and described in the paper. Quoting from the preface to the writer's printed thesis:

"The methods for computing secondary stresses have been greatly improved in recent years. Two objections, however, still remain: First, the amount of time involved is often excessive, and, second, the lack of a systematic checking device by which the correctness of the various steps of procedure may be ensured. While there are numerous other defects, these two alone are generally sufficient to reduce their practical utility. Ever since the beginning of 1917, when the writer undertook the analysis of secondary stresses in a two-hinged arch, the results of which have been published in the *Transactions* of the American Society of Civil Engineers, Vol. 82, p. 1104, it always occurred to him that there must be some method which is not only shorter and less cumbersome than the current ones but which also admits of a check. For two and a half years he had worked on the subject almost incessantly, striving to find some new method that will accomplish both the above mentioned results. At last, much to his satisfaction, the graphic method of deformation contour was obtained, which not only takes less time and checks itself, but also gives remarkably accurate results. Along with this method, almost contemporaneously, the analytic method was evolved."

It may be helpful to enumerate here the important advantages of these methods as compared with others.

Features common to both methods are:

(1) No simultaneous equations involving more than two unknowns are required.

(2) Both methods deal with a quantity which is of the same dimensional degree as the secondary stress itself. This gives more accurate results than when using a quantity involving large multiples of f.

(3) They take much less time than the ordinary methods.

(4) There are a number of checks available, especially for the graphic method.

(5) They can be easily applied to influence lines of secondary stresses.

(6) The complete information is contained in only one sheet of paper, including the rough computations, figuring, data, and formula.

Special features of the graphic method are:

(*a*) It is strictly graphical. From beginning to end no computations are required.

(*b*) It gives a complete representation of secondary stresses in every member of the truss.

(*c*) It not only shows secondary stresses at the two ends of a member, but also the variation of the stress along its whole length.

(*d*) It gives the point of inflection in a member and also the form in bending.

(*e*) With the aid of the force diagram for secondary stress lines it shows how the secondary stresses are affected by changing the section moduli of the members.

(*f*) It is more accurate than the ordinary analytic methods if the latter be computed by the use of a slide-rule. This is due to the fact that in analytic expressions the secondary stress is often found from the difference of two large numbers which must be very accurate, since the difference is small.

(*g*) The errors made in locating the base lines and vertex lines are not cumulative, as the effect of one line on the other is very small.

(*h*) It has a sufficient number of checks at every step of the procedure.

Characteristics of the analytic method are:

(1) In bridge offices where a large number of standardized trusses are to be analyzed for secondary stresses, the truss diagram, together with the small tables, could be blue-printed.

(2) This method is semi-graphical in that it shows the stresses at the proper places and their effects on each other.

(3) The entire process is mechanical. Every figure has a definite space; every procedure has a definite order.

(4) No figure is to be recorded twice and no necessary figures are omitted.

(5) All rough computations are shown on the diagram, thus rendering possible a check at any future time.

(6) The figures on the diagram are arranged so that all the additions and subtractions do not involve more than two lines of figures and can be accomplished without the use of extra paper, as when three columns of figures are to be added, etc. All the other computations can be made on a slide-rule.

(7) The method is capable of affording any degree of accuracy which may be desired.

(8) Practically all back references are eliminated.

(9) It has a unique check of secondary stresses as soon as they are found.

As explained in the paper both of the writer's graphic and analytic methods depend on two fundamental equations. The derivation of the deformation equation, as given in the writer's thesis, is criticized by the author as being unnecessarily complicated,* but the writer's aim was to derive it from Mohr's work equation with the hope that it might convey more physical meaning than a pure mathematical analysis. It appears complicated because it starts from the very beginning of the theory, but the principle involved is believed to be very simple, as the author agreed.†

The author also complains about the descriptions of the methods as being inevitably too long. This is due to the fact that many novel processes and features are introduced. Should some of the processes be already known to the engineers, there would not be so much explanation necessary. It can be easily seen that in Mohr's method, if one is not familiar with the Williot diagram or the Gauss solution, much more description would have been neces-

* *Proceedings*, Am. Soc. C. E., September, 1924, Papers and Discussions, p. 1009.

† *Loc. cit.*, p. 1024.

sary to complete the paper. The writer has aimed in the methods to systematize the procedure as much as possible, and this necessitates a detailed and minute description not only of the general principles involved, but also of the art of manipulation. This is deemed necessary to promote efficiency in the work and thus reduce the time and labor required. In his book on "Shop Management", Mr. Frederick W. Taylor gives a minute description of the most simple work of handling pig iron which, to most people, would seem to need no explanation. As it is acknowledged that secondary stresses require an enormous amount of time, any seemingly unnecessary yet important explanation would not be out of place in presenting a method that is considered perfectly new.

With reference to the writer's graphic method it is indeed gratifying to know that the author found it to give results that are "surprisingly accurate".* This is in agreement with what the writer himself proved when testing the method and later with the results obtained by his students at the Tangshan Engineering College. The author, however, expressed his unwillingness to recommend the method on the basis that all graphic methods are undesirable; his objections therefore will virtually exclude every graphic solution from practical use. First, he seemed to dislike the use of a large-sized drawing paper on the ground that it might become unwieldy. It is true that to solve accurately a long-span truss under unsymmetrical loading a sheet of unusually large size is indispensable; but this is not a feature peculiar to the writer's method alone—all graphic methods, without exception, are subject to this disadvantage. When a large-sized drawing paper is compared with ten or more pages of tabulated computations as used in other methods (the author used seven tables in the Manderla-Gauss method, requiring fully ten sheets of 8 in. by 12 in. paper), it is only a question of individual taste as to which is more adaptable to use. Personally, the writer much prefers the drawing to the tabulated sheets for the evident reason that it conveys a complete mental picture of the problem and thus enables the computer to see the effect of one stress on the other. This is important from the standpoint of design, as it will help to change the proportioning of the members so as to arrive at a more balanced design. When the designer is faced with a large mass of numbers, whether tabulated or not, he is really solving a mathematical problem and is at a loss to realize the significance of the result obtained in every step of the procedure. Inasmuch as secondary stresses are important on account of their practical bearing on the design, rather than of their mere mathematical interest, every method proposed for their solution should take this factor of illustration into account.

The author further objects to the graphic method because of the difficulty of adapting it to trusses with subdivided panels. This is true for the method presented in the present form, but the writer believes that it can be modified to accomplish the purpose, as it is certainly susceptible to improvements.

It is rather surprising that the author complains of the labor required to arrive at the "base" and "vertex" lines as "excessive". He did not give the

* *Proceedings*, Am. Soc. C. E., September, 1924, Papers and Discussions, p. 1074.

actual time consumed in getting these sets of lines; the writer, from personal experience in solving for a number of trusses, did not find it unbearable. To construct those two sets of lines would require some time, but it must be remembered that this corresponds to that used in forming and solving the large set of simultaneous equations as demanded by other methods like those of Manderla or Mohr. To determine whether or not the time or labor required is excessive, an accurate record of the actual time consumed for each method should have been kept, thus rendering possible a conclusive comparison. It is the hope of the writer that the author has made such a record in his work, as the time element is an important factor in the discussion of methods proposed for the analysis of secondary stresses.

In connection with the "base" and "vertex" lines, the author remarks that because of the many repetitions of the same process, the labor involved "is bound to become extremely monotonous".* In the writer's opinion, however, this feature of the method is something that deserves merit instead of criticism, as it is well known that to cultivate speed and accuracy "repetition of the same process" is highly desirable. It must be remembered that the subject of secondary stresses is important because of its practical bearing on the design; if it appealed only to the mathematician it would not have interested the bridge engineer. Most bridge engineers object to the consideration of secondary stresses largely because of the excessive labor required for their complete solution. Any means proposed to facilitate the solution is, therefore, in urgent demand. Such psychological effect as monotony, even though it may exist and be unpleasant, is indeed too trivial to mention if the method be adaptable to practical use.

The author seems to judge the writer's graphic method from his experience with other graphic solutions, for in discussing the question of checking devices, he states† that "any graphical method is objectionable in the drafting-room as a complete and thorough check by a second man requires an entirely new solution of the whole problem". It is true that if the second man's check is to make sure that every line in the drawing has its correct position, he will have to repeat exactly the same process as performed in the first solution; but if his purpose is to determine the correctness of the result, there are in the writer's method two neat and simple checks. In the first place (Fig. 57‡), there are two sets of "base" and "vertex" lines for the first triangle of the truss (Triangle A) each using one end of the hip vertical as the vertex. This gives the lines as marked "Triangle A, Vertex (2)" and "Triangle A, Vertex (3)." Now only one of these two sets is required in the solution of the problem while the other is not used until all the "correct deformation contours, r," are found. If the contours for the vertices of the first triangle satisfy the other set of base and vertex lines, not used in the solution, the correctness of every step in the procedure is thereby insured.§ (This is similar, in a way, to the check of an ordinary Maxwell stress diagram.) Another check of the solution is obtained by draw-

* *Proceedings*, Am. Soc. C. E., September, 1924, Papers and Discussions, p. 1074.

† *Loc. cit.*, p. 1074.

‡ *Loc. cit.*, p. 1068.

§ *Loc. cit.*, p. 1069.

ing equilibrium polygons around every joint of the truss. If the secondary stresses produce no moment around the joints, the correctness of the secondary stresses themselves is also proved (or the writer's graphic method of successive deduction may be used instead). It is evident, therefore, that the writer's graphic method is self-checking and there is no necessity of going over the whole process, even if it has to be checked by a second man.

The author questions* the use of two 45° lines with which to close the deformation contour. Although these lines are not essential to the solution of the problem, they are introduced for the purpose of furnishing an additional check, as their intersection point must lie on the deformation line constructed for the outside angle of the truss around the joint.

The value of the writer's graphic method will be greatly enhanced by using a drafting machine. The time saved may be compared with that in the analytic solution by using an adding machine.

The writer's analytic method was evolved for a twofold purpose: First, to afford a practical solution of the problem, reducing much of the labor and time required in comparison with the existing methods; second, to introduce a new arrangement of computation believed to be more rational and useful than the ordinary tabulated form. The principles involved in this analytic method are simple and are believed to be more understandable than most of the existing methods. Were it not for the fact that a novel form of computation has been evolved at the same time, intended to promote efficiency and reduce the chances of making mistakes, the description of the method would have been materially shortened.

The writer is much disappointed that the author has failed to mention some of the exclusive features of the analytic method, especially in regard to the amount of time required for a solution. The writer has found from repeated experience that under symmetrical loading the analytic method would require not more than one-half the time necessary in other solutions, whether the span is long or short. As full loading is probably the only type that will be investigated for secondary stresses in practice, in view of the fact that those stresses are now rarely investigated for any kind of loading, the writer's analytic method would supply a practical working tool for the modern bridge office. It is more rational and systematic than the others and requires only a few hours' work for a complete solution. When the bridge office has to design a large number of standard trusses the writer's method is even more useful, on account of the fact that the "computation diagrams" may then be blueprinted for permanent use.

Under unsymmetrical loading the author finds that the coefficients of the unknowns increase very rapidly as triangle is added to triangle. The writer is not unaware of this shortcoming of the method, as he has also tested the method for unsymmetrical loading, but on a closer study of the various elements involved, he does not consider it to be very serious. The reason why those coefficients seem to follow on an ever-increasing trend, is due to the peculiar way in which they are formed. Almost all the coefficients in the

* *Proceedings*, Am. Soc. C. E., September, 1924, Papers and Discussions, p. 1017.

U-tables are obtained by adding numbers derived from the previous coefficients. The major part of the arithmetic operations required, therefore, is addition, which is the easiest form of calculation even with large numbers. Subtraction of numbers is found in rare cases, while multiplication and division are necessary only in the moment equations around the joints and the checking equations in each triangle. With most of the arithmetic operations in addition, the writer cannot see that the practical value of the analytic method would be materially reduced even in a long-span truss.

The analytic method has a unique check which must be satisfied as soon as the stresses in a triangle are completely known; but, as the writer noted in the printed abstract of his thesis, "particular attention should be paid to the solution of equilibrium equations and also to the correct substitution of the values of U, as the checking equation does not apply in either case." In criticizing the checking feature of the analytic method, the author has probably overlooked that statement.

To those who are familiar with the writer's paper, it is probably not necessary to remark that the analytic method is also adaptable to trusses with subdivided panels. Whether the triangles of the truss have vertices in the interior or on the boundary of the frame, does not seem to have any effect on the utility of the method. A fair comparison can be made only by applying all methods to the same kind of structure.

It will be noted that in the analytic method the computations are arranged so that every operation has a predetermined order and all necessary calculations are clearly shown in the diagram, thus eliminating the personal equation and facilitating a thorough check at any future time. Another exclusive feature of the method is found in the fact that the unknowns of the problem are reduced to the secondary stresses themselves, which enables the computer to get some idea of their relative importance as soon as they are found. It is sincerely hoped that these features may provoke new ideas in the subsequent investigations of secondary stresses.

In 1922, two of the writer's students at the Tangshan Engineering College, Messrs. T. S. Yen and T. H. Kê, applied his process in a general way to Mohr's method, thereby arriving at two modified forms, graphic and analytic, which enabled them to solve the problem in much less time than with the original method. These modified forms were patterned very much after the writer's methods except that the angular rotation of the joints were used as unknowns instead of the deformation contours. This shows that a continued interest in this subject will broaden the scope for investigation and any step tending to promote the interest cannot be taken in vain.

The writer wishes to express his indebtedness to the Society for the publication of this valuable paper of Dr. von Abo, especially as nearly one-quarter of the printed matter is devoted to the writer's two methods. This worthy task undertaken by the Society will surely be deeply appreciated by the Engineering Profession at large.

DESIGN OF SYMMETRICAL CONCRETE ARCHES

Discussion*

By Messrs. William Cain and Carl B. Andrews.

William Cain,† M. Am. Soc. C. E. (by letter).‡—The analysis of symmetrical concrete arches, as presented by the author, is the most complete that has appeared. After deducing formulas that apply to any arch rib, a formula for the arch axis is derived, based on a certain assumption as to the increase of dead load from the crown to the springing line, which, the author claims, coincides "almost exactly with the dead load pressure line for practical bridge arches". As the subsequent development is based on this assumption, it is hoped that designers with sufficient data will give it a thorough test and publish their results.

The assumption leads to simple formulas for computing the co-ordinates of points on the arch axis and for ascertaining the dead load reactions.

A formula has been assumed, likewise, for the thickness of the arch; it gives a pleasing contour and leads by an extended analysis to formulas for the reactions, M', V_L, and H, for a live load, $P = 1$, at any point of the arch. The formulas have been checked by the writer. Their use is greatly facilitated by the many tables and diagrams given in the paper. Also, the diagrams giving maximum and minimum bending moments, with corresponding thrusts, for the crown, springing line, and quarter-points, together with temperature and rib-shortening effects, will be especially appreciated by the engineer.

Taking the origin at the elastic center simplifies the general formulas for any arch. Its main advantage is that it enables the solution to be readily effected by the use of equilibrium polygons, whether the arch axis has been divided into parts such that Δx is constant or such that Δw remains the same for each supposed voussoir. When Δx is taken as constant, the formulas involve a great deal of computation, so that the graphical method has been exclusively used hitherto.

However, when the axis is divided so that Δw is constant for the successive voussoirs the formulas can be reduced to simple forms, readily computed, and giving precise results which, afterward, can be checked by the approximate graphical methods. Graphical solutions are approximate, for each line drawn involves a possible error in length, direction, or intersection, so that the result rarely checks with the true position when ten or twenty lines have to be drawn.

* This discussion (of the paper by Charles S. Whitney, M. Am. Soc. C. E., published in November, 1924, *Proceedings*, and presented at the meeting of February 4, 1925), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

† Prof. Emeritus, Univ. of North Carolina, Chapel Hill, N. C.

‡ Received by the Secretary, December 26, 1924.

The simplified formulas will be deduced subsequently and applied to the particular arch, Fig. 19,* not only to illustrate the method of computation, but to compare results with those found by the author, where the division of the neutral axis was such that Δx was constant. The points on the arch axis given in Fig. 19 were plotted, the curve of the axis sketched in by eye, and its length measured and found to be 53.42 ft. A scale of 2 ft. = 1 in. was used. To this scale, lay off, in Fig. 57, $A\ C = 53.42$ ft. and draw the I-curve by the

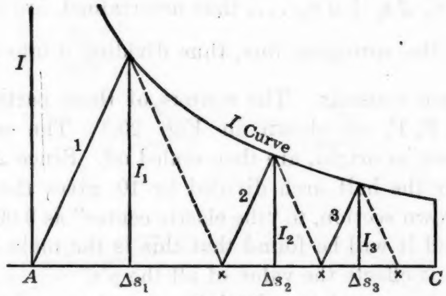


FIG. 57.

aid of Table 1,* taking for convenience, 0.001 of the tabular value. To divide the length, $A\ C$, from the springing line, A, to the crown, C, into ten parts, $\Delta s_1, \Delta s_2, \ldots, \Delta s_{10}$, such that:

$$\frac{\Delta s_1}{I_1} = \frac{\Delta s_2}{I_2} = , \ldots = \frac{\Delta s_{10}}{I_{10}} = \text{constant}$$

I_1 being 0.001 of the moment of inertia at the mid-point of Δs_1, etc., first assume Δs_1, measure I_1 at its mid-point, and compute $\frac{\Delta s_1}{I_1}$. Then, at the end of Δs_1, draw Line 2 parallel to Line 1 to an intersection with the I-curve, measure I_2, and compute $\Delta s_2 = \left(\frac{\Delta s_1}{I_1}\right) I_2$ and lay it off. By continuing thus to compute, $\Delta s_2, \Delta s_3, \ldots$, the dotted parallel lines need never be drawn, so that nearly half the error of the usual construction is avoided, which is an important practical consideration.

For the arch in question, on the second trial, Δs_1 was assumed as equal to 10.72 ft., and the construction gave for ten divisions, $\Delta s_1 + \Delta s_2 + \ldots + \Delta s_{10} = 53.80$ ft., or 0.38 ft. in excess of $A\ C$. Assuming a proportionate decrease in Δs_1, it must be shortened by $\frac{0.38}{53.8}$ (10.72) = 0.08 ft. Thus, for the third trial, Δs_1, will be assumed to have the length, 10.72 — 0.08 = 10.64 ft. When the excess is much larger, it is well to take Δs_1 smaller than this proportionate rule gives. To proceed: With Δs_1 equal to 10.64 ft., at $\frac{1}{2} \Delta s_1$, measure $I_1 = 20.4 \therefore \frac{\Delta s_1}{I_1} = 0.521$. Hence,

$$\frac{\Delta s_2}{I_2} = \frac{\Delta s_3}{I_3} = \ldots, = 0.521$$

* *Proceedings*, Am. Soc. C. E., November, 1924, Papers and Discussions, p. 1357.

Thus, as before, at the end of Δs_1, draw Line 2 parallel to Line 1, measure I_2, and compute $\Delta s_2 = 0.521\, I_2$. Continue thus to compute and lay off, Δs_3, Δs_4, ..., Δs_{10}, only drawing the full lines, and find $\Delta s_1 + \ldots + \Delta s_{10} = 53.40$ ft. The difference, $53.42 - 53.40 = 0.02$ ft., can now be distributed between any two divisions. It is hoped that the writer may be pardoned for entering into this detail, as the subject seems to offer difficulties to some, who spend an unnecessary amount of time on this division of the arch axis.

The lengths, $\Delta s_1, \Delta s_1 + \Delta s_2, \ldots$, thus ascertained, are laid off on the arch axis, beginning at the springing line, thus dividing it into parts such that $\frac{\Delta s}{I}$ is the same for each voussoir. The centers of these parts are then marked, 1, 2, ... 10, 10′, ... 2′, 1′, as shown in Fig. 20.* The co-ordinates, x, y_0, referred to the crown as origin, are then scaled off. Since Δw is constant, the sum of the y_0's for the half arch divided by 10, gives the distance below C, the center of the crown section, to "the elastic center" as 3.093 ft. The author's value is 3.14 ft., and it will be found that this is the main discrepancy in the two methods, since it effects the value of all the y's.

Taking the origin now at the elastic center, the x-axis horizontal and the y-axis vertical, $y = 3.093 - y_0$. The values of x and y for the points 1′, 2′ ..., 10′, are inserted in Table 21, together with their squares and products.

TABLE 21.

Point. (1)	y. (2)	y^2. (3)	x. (4)	x^2. (5)	$x\,y$. (6)
1′	−8.71	75.86	45.80	2 098	−398.92
2′	−4.23	17.89	37.94	1 439	−160.49
3′	−1.60	2.56	31.56	996	− 50.50
4′	+0.08	0.01	25.96	674	+ 2.08
5′	+1.21	1.46	21.00	441	+ 25.41
6′	+1.93	3.72	16.64	277	+ 32.12
7′	+2.46	6.05	12.58	158	+ 30.95
8′	+2.80	7.84	8.75	77	+ 24.50
9′	+3.00	9.00	5.15	27	+ 15.45
10′	+3.06	9.36	1.70	3	+ 5.20
		133.75	207.08	6 190	−609.91
		2		2	+135.71
....	$\Sigma_B^C\, y = 0.$	267.50		$\Sigma_B^A\, x^2 = 12\ 380$	$\Sigma_B^C\, x\, y = -474.20$

The formula for M' is:†

$$M' = P \frac{\sum_B^P x' \Delta w}{2 \sum_B^C \Delta w},$$

in which, $\Delta w = \frac{\Delta s}{E\, I}$ is constant. (See Fig. 58.)

* *Proceedings*, Am. Soc. C. E., November, 1924, Papers and Discussions, p. 1359.
† *Loc. cit.*, p. 1347.

Let

$N =$ total number of voussoirs in the arch;

$n =$ number of points, such as 1′, 2′..., between B and the load, $P = 1$;

$d =$ horizontal distance from the crown to P;

$x' =$ horizontal distance from P to points such as 1′, 2′, ..., to its right.

$$\therefore 2 \sum_B^C \Delta w = N \cdot \Delta w\,;\; x' = x - d$$

$$\therefore M' = \frac{1}{N} \sum_B^P x' = \frac{1}{N} \sum_B^P (x - d)$$

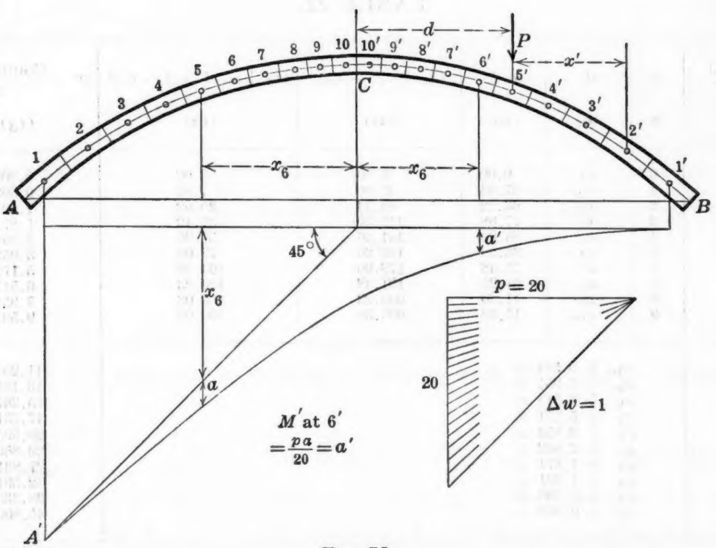


FIG. 58.

Designating the co-ordinates, x, y, of the points, 1′, 2′, ..., 10′, as (x_1, y_1), (x_2, y_2), ... (x_{10}, y_{10}):

$$\sum_B^P (x - d) = (x_1 - d) + (x_2 - d) + \dots + (x_n - d)$$

$$\therefore M' = \frac{(x_1 + x_2 + \dots + x_n) - n\,d}{N}$$

Thus, if $P = 1$ is placed successively at the points, 1′, 2′, ..., 10′, d takes the successive values, x_1, x_2, ..., x_{10}, and n, the successive values, 0, 1, 2, ..., 9. (See Fig. 58.)

No mistake can be made in writing the value of n if Fig. 58 is noted. It does not include the point where P is placed. In reality, P is a distributed load, so that in case the equilibrium polygons of Fig. 17* are drawn, they

* *Proceedings*, Am. Soc. C. E., November, 1924, Papers and Discussions, p. 1353.

should be rounded off to form a smooth curve to correspond to infinitesimal divisions. The same remark applies to the graph of any of the derived quantities.

If the equilibrium polygon, 4′, of Fig. 17 is drawn with the pole distance, p_4, equal to the sum of the loads, $\Sigma \Delta w$, the end tangents make an angle of 45° with each other, as in Fig. 58, and a vertical ordinate from the tangent at the right to the curve, to the scale of distance, is equal to M'. Thus, for a load, 1, at the point, 6′, $M' = a'$. For a load, 1, at the symmetrical point, 6, $M' = x_6 + a = x_6 + a'$, since $a = a'$. By this principle, from the values of M' already computed for the points, 1′, ..., 10′, the values of M' for the points, 1, ... 10, are derived. Table 22 gives the results.

TABLE 22.

Load at Point: (7)	n. (8)	d. (9)	$n\,d$. (10)	$(x_1 + \ldots + x_n)$. (11)	$(x_1 + \ldots + x_n) - n\,d$. (12)	$M' = \frac{\text{Column (12)}}{20}$. (13)
1′	0	x_1	0.00	0.00	0.00	0.000
2′	1	x_2	37.94	45.80	7.86	0.393
3′	2	x_3	63.12	83.74	20.62	1.031
4′	3	x_4	77.88	115.30	37.42	1.871
5′	4	x_5	84.00	141.26	57.26	2.863
6′	5	x_6	83.20	162.26	79.06	3.953
7′	6	x_7	75.48	178.90	103.42	5.171
8′	7	x_8	61.25	191.48	130.23	6.511
9′	8	x_9	41.20	200.23	159.03	7.951
10′	9	x_{10}	15.30	205.38	190.03	9.504
10	$x_{10} + 9.504 =$					11.204
9	$x_9 + 7.951 =$					13.101
8	$x_8 + 6.511 =$					15.261
7	$x_7 + 5.171 =$					17.751
6	$x_6 + 3.953 =$					20.593
5	$x_5 + 2.863 =$					23.863
4	$x_4 + 1.871 =$					27.831
3	$x_3 + 1.031 =$					32.591
2	$x_2 + 0.393 =$					38.333
1	$x_1 + 0.000 =$					45.800

The formula for V_L*, when Δw is constant, reduces to:

$$V_L = \frac{\sum_B^P x\,x'}{\sum_B^A x^2} = \frac{x_1^2 + x_2^2 + \ldots + x_n^2 - d\,(x_1 + x_2 + \ldots + x_n)}{\sum_B^A x^2}$$

since,

$$x' = x - d \therefore x\,x' = x^2 - d\,x.$$

The right vertical reaction for a load, $P = 1$, at 10′ is $1 - V_L = 1 - 0.4716 = 0.5284$. From symmetry, however, this is the vertical reaction at the left springing line for a load, $P = 1$, at Point 10. The values of V_L for a load, $P = 1$, at Points 10, 9, ... 1, are computed on this principle. Table 23 gives the results.

* *Proceedings*, Am. Soc. C. E., November, 1924, Papers and Discussions, p. 1346.

TABLE 23.

Load at Point. (14)	n. (15)	$x_1^2 + x_2^2 + \ldots + x_n^2$. (16)	$x_1 + x_2 + \ldots + x_n^2$. (17)	$d = x$. (18)	$d\,(x_1 + \ldots + x_n)$. (19)	Column (16) — Column (19). (20)	$V_L = \dfrac{\text{Column (20)}}{12\,380}$ (21)
1′	0	0.	0.00	45.80	0.	0.	0.0000
2′	1	2 098.	45.80	37.94	1 736.	362.	0.0292
3′	2	3 537.	83.74	31.56	2 645.	892.	0.0720
4′	3	4 533.	115.30	25.96	2 998.	1 535.	0.1240
5′	4	5 207.	141.26	21.00	2 967.	2 240.	0.1810
6′	5	5 648.	162.26	16.64	2 701.	2 947.	0.2380
7′	6	5 925.	178.90	12.58	2 254.	3 671.	0.2965
8′	7	6 083.	191.48	8.75	1 676.	4 407.	0.3560
9′	8	6 160.	200.23	5.15	1 031.	5 129.	0.4143
10′	9	6 187.	205.38	1.70	349.	5 838.	0.4716
10	1 — 0.4716 =						0.5284
9	1 — 0.4143 =						0.5857
8	etc.						0.6440
7							0.7035
6							0.7620
5							0.8190
4							0.8760
3							0.9280
2							0.9708
1							1.0000

The formula for the horizontal thrust is,* for $\Delta\, w$ constant and $P = 1$:

$$H = -\frac{\sum_B^P y\, x'}{\sum_B^A y^2}$$

When n (as hitherto) = number of points, such as 1′, 2′, ..., between B and P:

$$\sum_B^P y\, x' = \Big[y_1\,(x_1 - d) + y_2\,(x_2 - d) \;\ldots\ldots + y_n\,(x_n - d)$$

$$= x_1\, y_1 + x_2\, y_2 + \ldots + x_n\, y_n - d\,(y_1 + y_2 + \ldots + y_n).$$

For the points, 1′, 2′, ..., 10′, considered, x is always positive, but since y may be positive or negative, it is imperative that it should be given its proper sign, as shown in Table 21, from which,

$$\sum_B^A y^2 = 267.5$$

$$H = \frac{-\,(x_1\, y_1 + \ldots + x_n\, y_n) + d\,(y_1 + \ldots + y_n)}{\sum_B^A y^2}$$

The results are given in Table 24.

It will be observed that the formulas for M', V_L, and H, are very simple and direct, so that mistakes can be readily avoided with moderate care. The computations take little, if any, more time than the graphical methods, which can be used to check them. It will be noted, further, that the formulas only apply to the right half of the arch. The quantities for the left half are simply

* *Proceedings*, Am. Soc. C. E., November, 1924, Papers and Discussions, p. 1394.

derived, as shown from those pertaining to the right half. Similarly, it is necessary to draw the equilibrium polygons only for the right half of the arch.

TABLE 24.

Load at Point. (22)	n. (23)	$x_1 y_1 + \dots + x_n y_n$. (24)	d. (25)	$y_1 + \dots y_n$. (26)	$d\,(y_1 + \dots + y_n)$. (27)	− Column (24) + Column (27). (28)	$H = \frac{\text{Column (28)}}{267.5}$ (29)
1′	0	0.00	45.80	0.00	0.00	0.00	0.000
2′	1	−398.92	37.94	− 8.71	330.46	68.46	0.256
3′	2	−559.41	31.56	−12.94	408.39	151.02	0.565
4′	3	−609.91	25.96	−14.54	377.46	232.45	0.869
5′	4	−607.83	21.00	−14.46	303.66	304.17	1.138
6′	5	−582.42	16.64	−13.25	220.48	361.94	1.353
7′	6	−550.30	12.58	−11.32	142.41	407.89	1.525
8′	7	−519.35	8.75	− 8.86	77.52	441.83	1.652
9′	8	−494.85	5.15	− 6.06	31.21	463.64	1.733
10′	9	−479.40	1.70	− 3.06	5.20	474.20	1.773

To compare results with those found by Mr. Whitney, who made Δx constant in the division of the arch, the values of H, M', and V_L, were plotted for the x's corresponding and the quantities scaled off at the points 5 ft. apart horizontally, marked 1, 2, 3, ... 10, 9′, ... 2′, 1′, on Fig. 28.*

The results are given in Table 25, where Columns (1) refer to the writer's computations and Columns (2) to Mr. Whitney's values.

TABLE 25.

	H.		M'.		V_L.	
Point.	(1)	(2)	(1)	(2)	(1)	(2)
1	0.030	0.049	45.06	45.10	0.997	0.994
2	0.195	0.189	40.30	40.25	0.980	0.979
3	0.402	0.396	35.66	35.66	0.953	0.952
4	0.648	0.643	31.25	31.22	0.914	0.914
5	0.920	0.912	27.05	27.05	0.865	0.864
6	1.186	1.174	23.12	23.15	0.806	0.806
7	1.428	1.407	19.46	19.50	0.739	0.738
8	1.613	1.592	16.06	16.14	0.665	0.664
9	1.740	1.712	12.98	13.12	0.583	0.583
10	1.780	1.752	10.33	10.43	0.500	0.500
9′	......	......	7.98	8.12	0.417	0.417
8′	......	......	6.06	6.15	0.335	0.336
7′	......	......	4.46	4.50	0.261	0.262
6′	......	......	3.12	3.15	0.194	0.194
5′	......	......	2.05	2.08	0.135	0.136
4′	......	......	1.25	1.25	0.086	0.086
3′	......	......	0.66	0.67	0.047	0.048
2′	......	......	0.30	0.29	0.020	0.021
1′	......	......	0.06	0.08	0.003	0.006

Columns (1), $\Delta w = \frac{\Delta s}{E I}$ constant.

Columns (2), Δx constant.

* *Proceedings*, Am. Soc. C. E., November, 1924, Papers and Discussions, p. 1400.

On the whole, the quantities agree remarkably well. The differences are greatest with the load, $P = 1$, at the extreme point, 1′, as was naturally expected. The exact values have been determined by the author by integration. Thus, from Tables 15* and 16†, and from Figs. 31‡ and 32§ (for $m = 0.339$, $N = 0.186$), it is found, for the load, $P = 1$, at Point 1′, that,

$$M' = 0.068;\ V_L = 0.0051;\ \text{and}\ H = 0.045;$$

also, for $P = 1$ at Point 2, the exact method gives $H = 0.193$. All these quantities are intermediate in value to those corresponding to the two hypotheses, Δw constant and Δx constant, as given in Table 25.

For the usual combination of loads to give maximum moments, whether at the crown, springing line, or at any other point, the discrepancies, by either method, are negligible.

Horizontal Thrust Due to Temperature Changes.—From Equation (23 *a*)‖.

$$H = \frac{t\,\alpha\,l}{\int_B^A y^2 \frac{s}{E\,I}},$$

in which all lengths must be expressed in inches.

As l, s, and y, are in feet, and E in pounds per square inch,

$$H = \frac{E\,12\,l\,\alpha\,t}{\sum_B^A (12\,y)^2}\,\frac{I}{12\,s} = \frac{E\,l\,\alpha\,t}{12\sum_B^A (y^2)}\,\frac{I}{12\,s}$$

In dividing the arch axis into ten parts so that $\frac{s}{I}$ should be the same for each part, using the true value of I, it was found that,

$$s = 0.521\,\frac{I}{1\,000} \therefore \frac{I}{12\,s} = 160.$$

Substituting this value and $E = 2\,000\,000$, $l = 100$, $\alpha = 0.000\,006$, $\sum_B^A y^2 = 267.5$ (from Table 21),

$$H = 59.9\,t$$

and the moment at the crown,

$$59.9\,t \times 3.093 = 185.2\,t$$

Mr. Whitney's values are $H = 58.9\,t$, and the moment at the crown $= 185.0\,t$.

It is very gratifying to find that the two methods practically agree when only ten divisions of the half-arch are used for this arch of 100 ft. span. In this arch, the ratio of the radial thickness of the rib at the springing line to that at the crown is only 1.5. For a ratio of 3, or for a greater span than 200 ft., more than ten divisions of the half-arch should be used, possibly fourteen, to insure practical accuracy.

* *Proceedings*, Am. Soc. C. E., November, 1924, Papers and Discussions, 1382.

† *Loc. cit.*, p. 1384.

‡ *Loc. cit.*, p. 1403.

§ *Loc. cit.*, p. 1404.

‖ *Loc. cit.*, p. 1345.

With only ten points given the arch axis cannot be precisely sketched. A table showing the co-ordinates of twenty points on the axis, would give all the accuracy desirable for ordinary spans.

The thrusts from dead and live loads cause rib-shortening, inducing stresses which have too often been only roughly estimated. Thanks to the author more precise methods are now available with diagrams to aid in the computation of such stresses.

The author has recently published an illuminating article* on the special methods adopted by M. E. Fressynet, the noted French engineer, to reduce shrinkage, rib-shortening, and abutment deflection in concrete spans of nearly 600 ft. Fressynet's statements regarding the shrinkage effect on actual bridges are quite startling. As Mr. Whitney puts it, "it will be seen that for the ordinary 1 : 2 : 4 mix the shrinkage effect may be as great as the effect of a drop in temperature of from 60 to 90° Fahrenheit."

It is evidently much to be desired that a thorough investigation of this subject should be made, not on blocks of concrete, but on actual reinforced concrete arches. The effect of shrinkage is equivalent to that of a drop in temperature, but the amount has hitherto been taken as much less than Fressynet's observations would indicate.

Formulas for the arch subjected to horizontal loads have been derived; but they are not quite as simple as those given for vertical loads, as more than the half arch has to be considered in deriving them. They will not be given since horizontal loads are rarely considered; besides, there is a simple graphical method available.

The demonstration of Maxwell's theorem of reciprocal displacements, as given by the author, is very complete. However, in the application of these theorems, only the case of the arch fixed at the right end and free at the left end, is considered. For this case, a simple demonstration will be given here, by aid of the well known Equations (8) and (10).† Thus, consider the arch rib of Fig. 59, fixed at the right end, free at the left end, and subjected to a single horizontal load, $P = 1$, at the point, A. The case is quite general since y and y'' may be positive or negative.

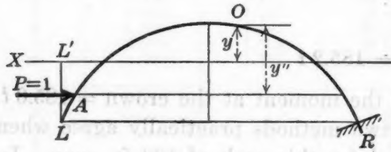


FIG. 59.

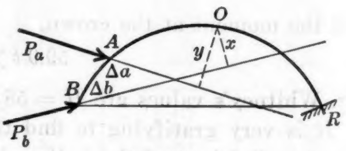


FIG. 60.

Regard L' as the free end; then, for any point, 0, y, being always reckoned from the free end, is as shown on Fig. 59. For the load, $P = 1$, the moment, M, about Point 0 is, $-1\, y'' = -\, y''$; hence, the horizontal displacement of the free end, L' is:

$$\Delta x = \int_A^R M\, y\, d\, w = -\int_A^R y''\, y\, d\, w$$

* *Engineering News-Record*, September 18, 1924, p. 463.

† *Proceedings*, Am. Soc. C. E., November, 1924, Papers and Discussions, p. 1337.

Next, consider the load, P, to be removed, and that a single horizontal force, $H = 1$, is applied at L'. This gives at 0 the moment, $M = -1\,y = -y$. As the horizontal deflection, $\Delta x'$, of Point A is desired, regard Point A as the free end; then, since the ordinates must be reckoned from the free end, y'' takes the place of y in Equation (10), giving the horizontal deflection of A, due to the load, $H = 1$,

$$\Delta x' = \int_A^R M\, y''\, d\, w = -\int_A^R y\, y''\, d\, w$$

since the deformation of the portion of the rib, $R\,A$, alone causes the deflection at A. This being exactly the same expression as was found for Δx, $\Delta x = \Delta x'$; or, the horizontal displacement of L' due to $P = 1$ at A is equal to the horizontal displacement of A caused by the horizontal force, $H = 1$, at L'. This is the first of Maxwell's theorems. The same result follows from replacing the sign of integration by that of summation, for the sum of the products, $y\, y''\, \Delta\, w$, from A to R, is the same in whatever order the factors are written.

From Fig. 6* and Equation (8), the rotation of the tangent to the neutral line at the free end, L, of the arch rib, Fig. 59, due to the load, $P = 1$, at A, is:

$$\Delta\,\phi = -\int_A^R M\, d\, w = \int_A^R y''\, d\, w\dagger$$

Now, suppose that P is removed and that a couple of moment, $M' = 1$, acts at L. Then, regarding A as the free end, the horizontal displacement, Δx, of A is, by Equation (10):

$$\Delta x = \int_A^R M'\, y''\, d\, w = \int_A^R y''\, d\, w,$$

since y'' is now the ordinate referred to the supposed free end, A $\therefore$ $\Delta\,\phi = \Delta\, x$. This proves the second of Maxwell's theorems for the rib fixed at one end: That the rotation of the tangent at L in radians caused by the force, $P = 1$, at A, is equal to the linear displacement of the point, A, in the direction of P, produced by the couple the moment of which is $M' = 1$ acting at L. The third of Maxwell's theorems is similarly easily proved.

A more general demonstration can be given by supposing two inclined unit forces, P_a at A, and P_b at B, Fig. 60, to act alternately on the arch rib, fixed at the right end and free at the left end.

When P_a alone is acting, suppose the deflection of B in the direction of P_b to be $\Delta\, b$, and when P_b alone is acting, suppose the deflection of A in the direction of action of P_a to be $\Delta\, a$. To prove that $\Delta\, a = \Delta\, b$, use will again be made of Equation (10), noting that y is the ordinate of O with reference to the free end and that it is always perpendicular to the direction of the deflection of this end. This likewise can be easily proved directly.

* *Proceedings,* Am. Soc. C. E., November, 1924, Papers and Discussions, p. 1337.

† Referring to Fig. 6 and Equation (8) (*Proceedings.* Am. Soc. C. E., November, 1924, p. 1337) it may be noted that when M, the moment of the loads on the part of the cantilever from A to (x, y) about (x, y), is negative or counter-clockwise, that $\Delta\,\phi$ is positive representing an increase in the angle; hence, to agree with this convention as to signs, Equation (8) should be written, $\Delta\,\phi = -\int_A^B M\, d\, w$. If Equations (9) and (10) are used as written, then, when M is negative, A moves to the right and downward, as in the diagram; when M is positive, A moves upward and to the left.

In Fig. 60, let x and y be the perpendiculars from O to P_b and P_a produced, respectively. Thus, x is perpendicular to the direction of Δb and y is perpendicular to the direction of Δa.

When $P_a = 1$, alone acts on the rib at A, then regarding B as the free end, having a deflection, Δb, in the direction of action of P_b, x perpendicular to Δb must replace y in Equation (10). Likewise, $M = -P_a\, y$. Hence, using summation in place of integration:

$$\Delta b = \sum_A^R M\, x\, \Delta w = - \sum_A^R x\, y\, \Delta w.$$

Next, suppose that P_a is removed and that $P_b = 1$ at B alone acts on the rib, causing the deflection, Δa, at A in the direction of action of P_a. Now, y perpendicular to Δa replaces the y of Equation (10), and $M = -P_b\, x = -x$. The deformation of the portion of the rib, $R A$, alone causes the deflection at A, so that the integration will only extend from R to A.

$$\therefore \Delta a = \sum_A^R M\, y\, \Delta w = - \sum_A^R x\, y\, \Delta w.$$

It follows that $\Delta a = \Delta b$, which proves the first of the Maxwell theorems.

In summing from R to A, x will be positive or negative according as O is above or below the line of action of P_b; similarly for y, if the line of action of P_a produced passes above R.

Now consider the rotation, $\Delta \phi$, of the tangent to the neutral line at B when $P_a = 1$ alone acts. This rotation is due to the deformations in the rib from R to A and is the same at B as at A. Then $M = -P_a\, y = -y$, and by Equation (8):

$$\Delta \phi \text{ (at } B) = - \sum_A^R M\, \Delta w = \sum_A^R y\, \Delta w.$$

Next, suppose P_a to be removed and that P_b at B is replaced by a couple the moment of which is $M' = 1$. This couple, acting alone on the rib, produces a deflection, Δa, at A in the direction of action of P_a. Taking A as the free end, as before,

$$\Delta a = \sum_A^R M\, y\, \Delta w = \sum_A^R y\, \Delta w$$

$$\therefore \Delta a = \Delta \phi$$

giving the second of Maxwell's theorems. The third theorem follows, because for the deformations in $R A$ only, $\Delta \phi$ at A and B must be the same.

As far as the writer knows, the preceding demonstrations are new.

In conclusion, the writer thinks that the author has given a very valuable paper—one that has added materially to present knowledge of arch bridges and that should lead to improved designs.

CARL B. ANDREWS,* ASSOC. M. AM. SOC. C. E. (by letter).†—The author's proposal to make I inversely proportional to $\cos \phi$ is a departure from the previous general rule that the radial thickness of the arch rib should be inversely

* Prof. of Eng., Univ. of Hawaii, Honolulu, Hawaii.

† Received by the Secretary, December 29, 1924.

proportional to cos ϕ. The old rule would make the vertical projection of all radial sections equal; the author's suggestion would result in the third power of the radial thickness being inversely proportional to cos ϕ. In the case of arches with considerable rise, the ribs might be rather thin. In this connection, the equation:*

$$d\,w = \frac{d\,s}{E\,I} = \frac{\cos\phi\,d\,x}{E\,I} = \frac{b_1\cos\phi}{E\,I}\,d\,z,$$

seems to be wrongly stated. If $d\,s = \dfrac{d\,x}{\cos\phi}$, it should read,

$$d\,w = \frac{d\,s}{E\,I} = \frac{d\,x}{E\,I\,.\cos\phi} = \frac{l_1\,.\,d\,z}{E\,I\,.\cos\phi}$$

If then I is made inversely proportional to cos ϕ, the quantity, I . cos ϕ, can be eliminated.

It is evident that the author has done an immense amount of work in the preparation of this paper, for which he deserves the thanks of all engineers interested in arch design. The formidable appearance of his demonstrations, however, is somewhat against the general adoption of his method. Engineers are prone to think that a single method of arch analysis is enough for one man to master in one life time, and are slow to take up a new method which looks just as hard as the old one to which they are accustomed.

---

* *Proceedings*, Am. Soc. C. E., November, 1924, Papers and Discussions, p. 1374.

FACTORS IN THE ZONING OF CITIES
A SYMPOSIUM

Discussion*

By Messrs. W. W. Crosby, Arthur S. Tuttle, Charles Wellford Leavitt and Max Wakeman Weir, W. A. Weldin, Rudolph P. Miller, Leonard S. Smith, Albert P. Allen, and E. P. Goodrich.

W. W. Crosby,† M. Am. Soc. C. E. (by letter).‡—The splendid paper of Mr. Knowles§ furnishes inspiration as well as fact that is of interest to all, particularly to engineers. Those of the latter who cherish the advancement of the profession can but admire the breadth and temper of his presentation and will receive from it encouragement for their further efforts. Even though engineers are now united for zoning, the education of other professional groups and of the "man in the street" is, as Mr. Knowles points out, fundamental for progress. The responsibility for this educational work must rest primarily on engineers, and such papers as this undoubtedly help the cause substantially.

Zoning is a form of co-operation. Healthful co-operation depends on the proper recognition of the community of interest. By intelligent presentation to the Society (as in this paper) and by comment on it by those so well qualified as are its members, interest and education in the matter should glow and spread. "On the anvil of Discussion are the sparks of Truth struck out." It is to be hoped they may fly far enough outside to do good.

Mr. Knowles suggests the need for the cultivation of support by the Courts for zoning principles and practice; he is quite right. Possibly a judicial recognition of nuisances other than those to the nose and ear will be thus induced to the great benefit of all.

He exposes briefly a rich "lead" if not an actual "mother-lode" in his remarks under "Skyscrapers".‖ The menace to the individual, singly and in incoherent groups, from this truly American development of a Frankenstein conception is not appreciated sufficiently until viewed both through a glass crystallized from a strong solution of community interest and in the light of zoning accomplishment. Then the menace becomes plain enough to inspire proper safeguards. At the same time such reasonable uses of the monster should be allowed as will be beneficial to the majority.

* This discussion (of the Symposium on Zoning presented at the meeting of the City Planning Division on October 24, 1924, and published in February, 1925, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Location Engr., State Dept. of Highways, Harrisburg, Pa.

‡ Received by the Secretary, October 7, 1924.

§ *Proceedings*, Am. Soc. C. E., February, 1925, Papers and Discussions, p. 178.

‖ *Loc. cit.*, p. 190.

Zoning is a practical recognition and application of the principles of democracy on which the United States is based. Democracy means community interest and the good of the majority—not abstract ultimate liberty for the individual and his personal desires or whims irrespective of those of his neighbors and fellow citizens. Belief in American principles must mean belief in and support of zoning; in the right of communities to zone; in the establishment of proper principles and practice for zoning; eventually these must result in a majority recognition of the benefits to be derived by both the public and the individual—either directly or indirectly—from zoning.

ARTHUR S. TUTTLE,* M. AM. SOC. C. E. (by letter).†—Mr. Knowles has brought out the basic principles on which zoning regulations must rest in order to stand the tests of public endorsement and litigation.

The municipal authorities of New York took a pioneer step in 1916 when they adopted the first comprehensive zoning plan. In no other city are property values so high. The fact that the Act has been sustained by the Courts in every particular is conclusive evidence of the soundness of the principles used. At that particular time, real estate was inactive, and values, in many sections, were seriously menaced by reason of the helter-skelter methods used by individual builders in developing their holdings to suit their own particular needs.

The zoning principle was welcomed by nearly every interest as the only means of restoring stable values, notwithstanding that in many instances its application to particular holdings often carried with it a temporary depreciation in value more or less serious in extent. Only three districts were recognized for use restrictions—residence, business, and unrestricted; five districts for height—1, 1¼, 1½, 2, and 2½ times the width of the street on which a building has frontage, with 50 ft. considered as the minimum width and 100 ft. as the maximum; and five districts for area—designated A, B, C, D, and E.

Since the adoption of the plan, and more particularly during the active period of building following the close of the World War, the discussion of zoning in New York has been continuous, but it has been confined almost wholly to particular districts where changes have been sought. As stated by Mr. Knowles, those which have been favored in the majority of instances are in the nature of increases rather than decreases in the restrictions.

The resolution of 1916 has been subjected to only a few amendments, and most of these are of a minor character, the only exceptions of note consisting of adding three new height districts, namely, a ¼-times district, a ½-times district, and a ¾-times district, to meet special conditions where exceptional treatment was deemed essential, of imposing a 10-ft. set-back in the E District, and of adding an F District designed to insure detached house development, coupled with the imposition of a 15-ft. set-back.

Under the law, the Board of Estimate and Apportionment is charged with responsibility for setting up and amending the zoning resolution and for

* Chf. Engr., Board of Estimate and Apportionment, New York, N. Y.

† Received by the Secretary, October 14, 1924.

applying it to areas of substantial size. Cases relating to the treatment of small parcels are left to a Board of Appeals. The Board of Estimate and Apportionment must give a public hearing, after due notice, concerning a proposed change either in the rules or in the plan, and in no case may action be taken until the merits of a proposed change have been investigated and reported upon by its Chief Engineer and given independent consideration by its Committee on City Plan. That the original plan has been so closely adhered to and that it has been sustained in every particular by the Courts is a strong tribute to the work done by the Committee in charge of the development of the plan as well as to the conservative method which has since been invariably observed by the municipal officers responsible for its maintenance and enforcement, and this notwithstanding the strong efforts that have been made to secure changes which would have introduced additional restrictions of considerable merit, but of a questionable character from a legal standpoint.

The allowance of a private garage for five cars in a residential district has occasioned much discussion, but the objections have been met in some measure by enforcing the provision against rental of any surplus space.

In the zoning of residential streets it has usually been the practice to exclude the frontage for a distance of 100 ft. from the intersection with business streets, with the expectation that the business development would front upon the latter. Repeated violations of this implied condition, with a prejudicial effect on high-grade residential property, has resulted in the serious consideration of a comprehensive re-study of all such areas to the end that a remedy may be found either through an amendment of the rules or through re-zoning. It will be recognized, of course, that property values on a business corner might be seriously affected if the lot subdivision were such that frontage could only be obtained on the business street and an attempt were made to enforce a residential use.

Before zoning was introduced, the history of the older streets which had grown up as residential, showed that as buildings of a substantial character became old-fashioned or out of keeping with the vicinity, it was the custom to convert them into other and more profitable uses, pending the time of demolition, and thus to secure the maximum revenue they might yield. Stores and industry were thus introduced into what were once exclusive residential areas, and, once introduced, the effect was to speed a similar change in character throughout the section. Since the introduction of zoning in New York, decay of this sort has been checked, and property owners are forced either to renovate or reconstruct such buildings unless an actual change in zoning is deemed meritorious. There is no question but that when the time for demolition has not clearly arrived and the buildings are out of keeping with what is required by the tenants, a serious loss is faced by the owner; but, on the other hand, if the zoning is not respected as reasonably permanent, it becomes only a question of time when all high-grade property must give way to less restricted uses.*

The symbols on the plans as originally adopted were placed within the street lines, and their interpretation in many cases could only be made by an

expert. In preparing a new edition of the maps, which has just been approved, advantage has been taken of the opportunity to substitute new symbols, placed within the block lines, showing clearly the exact boundary of the various districts—a method possessing marked advantage over the one earlier in use and conforming with that generally observed elsewhere.

If any one can doubt the merits of a proper zoning plan, it is only necessary to point out the amazing record of its acceptance throughout the United States as noted by Mr. Knowles. If kept within limits designed to recognize the necessity for light and air, and the general welfare of the community, and if considered along broad lines and within common-sense limits, there is now no question but that zoning can be legally sustained, and any tendency to introduce questionable or extreme requirements should be deprecated.

CHARLES WELLFORD LEAVITT,* AND MAX WAKEMAN WEIR,† MEMBERS, AM. SOC. C. E. (by letter).‡—Mr. Knowles' paper is clear and concise. Any further discussion compels a more or less re-statement of the matters already so ably set forth.

The right of eminent domain of the sovereign political subdivision has been invoked successfully for public improvements for many years. Its necessity is well recognized. It is exercised to secure to the public such lands as the public requires for parks, transportation, schools, streets, roads, public buildings, etc. Similarly, the police power has been exercised to secure to humanity, safety, health, light, and air.

These two well known and thoroughly acknowledged public rights or powers must not be confused or transposed. They have been established for two separate and distinct purposes: First, to obtain land for public use; and, second, to obtain public control over private activities to the limit of public safety and health.

To illustrate: A zoning ordinance with restrictions as to front-yard depth or set-back lines, may be drawn for some particular built-up community suffering from the traffic ills occasioned by narrow streets. The purpose is obvious. This method of securing the additional street width by invoking the police power is a confusion of the right of eminent domain—it seeks to deprive the owner of the proper use of a part of his land without just compensation. The Courts say "No, this shall not be done."

To illustrate how set-back restrictions can be applied under the police power, let a new development be laid out into streets and building lots and the lots sold with restrictions as to the set-back covering the whole development. The Courts say this may be done because all are participants, and the available lot area is known.

In the first illustration, the owner would be damaged because his lot area would be reduced, while his neighbor on a wider street would lose nothing. Further, the loss sustained by the damaged party would go to enrich the public domain. Engineers interested in zoning will do themselves and their

* Civ. and Landscape Engr. (Charles Wellford Leavitt & Son), New York, N. Y.

† Cons. Engr., New York, N. Y.

‡ Received by the Secretary, October 22, 1924.

clients a great injustice unless they keep in mind a clear conception of the distinction between these two different rights of the public.

In preparing a zoning ordinance and map, the comparatively common error of trespassing on the field of private restrictions must be avoided if success is sought. It must be kept in mind constantly that the police power can be invoked only on the grounds of safety, health, light, and air. The case of Nutley, N. J., illustrates this point very clearly. This case involved the right of use and came about through the opening of a retail store in a residence district. The decision of the Court was that there was no question of safety, health, light, and air involved and, therefore, such zoning against the use of property for a store was an improper invocation of the police power.

The welfare of the whole community must be the paramount consideration and no zoning should be attempted which, in any way, has for its object the special protection of any (so-called) class of people from the activities of another class. It must be remembered that the Courts recognize no classification of persons, but look on all as equal in the light of the statutes.

The education of the public and of the public official must be accomplished before great progress in zoning and in the administration of zoning ordinances can be expected. This education must be carried to the people interested, not in technical language such as engineers use one to another, but in language and thoughts that the people understand and appreciate, copiously illustrated with examples. It must be kept in mind that the problem of zoning is new to this generation, and needs study. Public and administrative officials should be taught not to be discouraged by adverse Court decisions. Every one who accomplishes anything will make some mistakes. The public and its officials must be taught to meet conditions with fortitude and take steps immediately to correct errors, without discouragement. It may seem that this is asking too much of an engineer. Every man who dedicates his life to a profession should know that his paramount duty is to improve the conditions of the people by education.

To be successful, zoning should meet the requirements of the enlightened public, should be helpful to all classes, applicable and adaptable to the community at the time, and permit adaptation to changed conditions at a future time. Care must be exercised in zoning that orderly, sane, competitive effort in the industrial and business growth is not throttled. Economic forces are inexorable and will not tolerate contravening Utopian ideas. Furthermore, the street system of a city will play an important part in the success or failure of practical zoning. For if zoning over-centralizes industry or business, the streets will become over-congested and transit facilities overtaxed, bringing burdensome and unnecessary expense upon the land, and creating inflated values from which no reasonable return can be obtained.

It is the opinion of many that zoning and city planning are so interlocked and closely allied as to be inseparable. For it is plain that to zone without thought of accessibility and intercommunication would be a fault not easily overcome. On the other hand, to plan without thought of adaptable use of the land would be only the wildest guess. Zoning is not a panacea for all the ail-

ments that a populous community is heir to, but, properly exercised, it will help a great deal.

Zoning a city before the city plan is devised may mean the eventual necessary abandonment of the best solution of the city plan, or it may lead to excessive costs in order to put into execution the best solution for the plan of a city.

Zoning may bring disaster to a city by congregating certain like industries, and thus developing a peak load on inadequate streets, instead of distributing the load by having diversified industries in each section of the city. By the latter means decentralization may be naturally secured. Thus, zoning may best be done after, or as a part of, the general city plan and not before, or independent of the city plan of which it is simply one small part. It might be more dangerous than establishing one of the other elements of city planning, such as traffic, transit railroad terminals, streets, parks, playgrounds, etc., without taking the others into consideration.

Zoning is invoked to stabilize values; perhaps it will for a time. The stabilization of values means that the price of land will not decrease materially, neither will it increase materially. While this effort is going on, however, what is to prevent the economic forces from becoming active because of some obscure happening, like the relocation of a railroad terminal. Stabilization of real estate values never will be accomplished by zoning any more than will the price of any commodity. It will always be controlled by demand. Zoning is not for the exclusive benefit of the land owner or speculator; it is for the whole people, and for no favored class. Its prime purpose is to help the public obtain the benefit of an efficient, full functioning development, with the least expenditure of time and money.

W. A. Weldin,* M. Am. Soc. C. E.—Pittsburgh engineers have lived long enough under the provisions of the Pittsburgh zoning ordinance to appreciate its excellence and the good work Mr. Knowles has done in drawing up the legislation and in setting up the machinery of enforcement and appeal. The ordinance seems to be popular, or, at least, the general idea of zoning is popular, and it seems to the speaker that, in spite of the large number of appeals, the legislation is a success.

It occurred to the speaker that there is a small detail in the matter of administration that has apparently been overlooked. In the Pittsburgh zoning ordinance, cemeteries are prohibited in residential sections by omission from the list of permitted uses of land. Obviously, the enforcement of this prohibition cannot be lodged with the building inspector. Some other agency will have to be called on. Perhaps the solution is with the Department of Public Health, which must give a burial permit. Perhaps this permit could be made to specify the place of burial, and so give some officer a chance to prevent illegal extensions of cemeteries.

It seems to the speaker that the large number of appeals in Pittsburgh is accounted for by the rugged topography, which creates so many difficulties for the builder. On this account, it is necessary to give the Board of Appeals

* Civ. and Min. Engr. (Blum, Weldin & Co.), Pittsburgh, Pa.

a wide range of discretion. This is a point that should not be forgotten in drawing up zoning ordinances.

Planning commissions and similar bodies have been much criticized for delay in rendering decisions. This is perhaps the greatest source of antagonism, especially on the part of those who are engaged in subdividing land, and in building. They feel they are the ones who are actually building the city (probably this is true as to the actual determination of what is to be built), and that they should be encouraged and helped, and not be subject to restrictions and delays. Delays in many cases will absorb a large part or all of the profits of an undertaking. This criticism does not apply to the Board of Appeals of Pittsburgh, which is acting promptly on whatever comes before it.

The information on the fire hazard in tall buildings given by Mr. Crane* is interesting and would have been still more so had it presented data as to the actual incidence of fire in such structures. Unless it can be shown that fewer fires, or smaller loss of life and property occur in 10-story buildings of modern type than in tall buildings, the alleged difficulties of fire-fighting would seem to be without application to the point under discussion.

In regard to the physiological effect of tall buildings on the inhabitants of a city, it seems to the speaker that Mr. Crane's assumption that dark and poorly ventilated rooms are adjuncts of the skyscraper is unwarranted. On the contrary, the tall buildings give a maximum of light and air for the workers therein. The great advantage to the health of the students offered by the proposed 52-story building of the University of Pittsburgh, as compared with the alternate scheme of 14 acres of 4-story buildings, is one of the strong points urged in favor of the tall building project. That a reduction of light and air on the street surface makes the streets "disease-breeding canyons" seems a rather exaggerated statement.

As to economies of the tall building there are too many of these structures built by banks and estates from trust funds, to substantiate the fear that such investments are deliberately and repeatedly made only to produce an annual net loss. The University of Pittsburgh project is said on the authority of well-known engineers to result in a sufficiently lower operating cost to more than offset the slightly greater construction cost of the high over the low buildings.

If the speaker is correctly informed, buildings of twenty stories and more are being financed in several cities by bonds, which are redeemed, principal and interest, serially out of rentals. The mere fact that such bonds are thus paid off would seem to dispose of the "losing venture" theory. Besides, if the very high building is an economic fallacy, would it not be simpler to publish the figures, to stop further construction, than to legislate against it?

It seems to the speaker, that the ideal to be approached is tall buildings properly spaced, rather than dense masses of lower heights. The indicated legislation, then, would aim to prevent the very uniformity that a mere height limitation would create.

Cannot an ordinance be drawn, extending the principle of the New York law so as to require a set-back not only on the street front, but along all prop-

* *Proceedings*, Am. Soc. C. E., February, 1925, Papers and Discussions, p. 194.

erty lines; in other words, to permit a tall building only in case the prospective builder owns a lot of specified greater area than the base of the proposed tall portion?

RUDOLPH P. MILLER,* M. AM. SOC. C. E.—Most of those who discuss zoning treat it as something new. Mr. Edward M. Bassett, who is the leading authority on zoning, in a recent paper on its constitutionality, speaks of it as a new science. It should be considered not as new science, but rather as a new application of an old principle. The principle has been applied for many years in connection with the establishment of so-called fire limits as far as the construction of buildings is concerned, that is, districts in which the erection of frame buildings is prohibited. In this respect the principle has been sustained by the Courts in many decisions. It seems surprising, therefore, that the Courts at the present time are regarding zoning as a new principle. There have been some cases in which the Courts have held that a city could not fix fire limits unless it had been given specific authority to do so by statute, but since that time the principle seems to have been well recognized as an established practice, even without special authorization by statute.

Adverse Court decisions with respect to zoning have been largely because, under the guise of zoning, attempts have been made to do things that cannot be considered as in the interest of the public health, safety, or welfare.

Furthermore, too much differentiation is attempted in some zoning ordinances, even between one-family and two-family houses. There was a case recently in New York State in which an attempted differentiation was set aside by the Court. A sorority building was to be built in a residence district. The enforcing officer felt that a sorority could not be considered as a family. The Court held that there was no more harm in the erection of a sorority building in a residence district than of an ordinary dwelling. There would even seem to be some doubt as to whether a differentiation between one-family and two-family houses and multi-family houses will stand the test, although sentiment generally is so strong in favor of such distinction, that the Courts would probably take the public view of it.

Unless it is established that a given use in a district is clearly a nuisance, the Courts will not sustain its prohibition. In some cases, it has even been necessary to wait till the use has been established and proved to be a nuisance before the Courts would restrain its continuance in any particular district.

On every board of appeals, or board of adjustment, there should be an engineer. The personnel of such a board, however, should not include any official or any person who is connected with the formulation of the rules or laws, nor one who is engaged in the enforcement of those rules. An appellant before the board, if he must face the same officer who has refused him the permit, feels that his case is prejudiced at the start. There is no reason, however, why the enforcing officer should not appear before the board to state fully his position in the matter.

As to the limitation of height, it has been suggested that provision ought to be made to allow a building here and there to be erected to a considerable

* Cons. Engr., New York, N. Y.

height above the others. The New York law fairly well provides for this in the section which allows the erection of towers to unlimited heights, provided that in plan they are not more than 25% of the area of the lot. This makes it possible for two or more owners to combine, as they sometimes do, in the erection of buildings, and to erect a tower of considerable area and unlimited height, or any height that might be considered economically sound.

LEONARD S. SMITH,* M. AM. SOC. C. E.—The remarkable growth of city planning and zoning in American cities is undoubtedly the outstanding civic development of the present day. The realization of these plans, however, has been greatly handicapped through a lack of understanding by the general public with a resulting fatal lukewarmness toward carrying them out. Many a city has a plan, but lacks the necessary leadership to act on it.

The great universities are taking an important part in supplying this needed understanding and future leadership; for example, at the Engineering College of the University of Wisconsin, courses in City Planning have been given for about fifteen years. During the last six years, 1 250 students have taken one or more of the three courses offered. More than half these students have made investigations of the city planning needs of their home towns and have incorporated them in well prepared and illustrated themes. In addition to engineers, these students included those having economics, sociology, journalism, and landscape architecture as their major subjects.

For the most part they become much interested in city problems, and judging from letters received by the speaker, continued their interest after graduation. It would not seem to be a vain hope to visualize many of these men and women as future leaders of civic improvement in their home cities; in fact, examples of this have already been noted.

Probably many other universities are making similar records and, if so, the aggregate influence of these graduates will eventually be an important factor in the problem of getting things done at the right time and place.

The obvious need for engineers to take a greater interest in civic movements and to furnish leadership has been emphasized by numerous writers. In no better way can the profession secure a well earned appreciation than by giving this type of public service.

ALBERT P. ALLEN,† ESQ.—The telephone business is one of perhaps a small group whose daily bread may be said to depend on the unanswered question of zoning and its actual solution, either with or without proper ordinances. In other words, for many years, this industry has been trying to anticipate what the actual zoning in Chicago was going to be; this also is true in connection with all the principal cities in Illinois. For reasons indicated in the speaker's paper,‡ a telephone company has to look further ahead in connection with the distribution and character of population than the other utilities, and it is a fact that the work it is now approving and starting is dependent to a

* Prof. of City Planning and Highway Eng., Univ. of Wisconsin, Madison, Wis.

† Commercial Engr., Illinois Bell Telephone Co., Chicago, Ill.

‡ "The Influence of Zoning on the Design of a Telephone Plant", *Proceedings*, Am. Soc. C. E., February, 1925, Papers and Discussions, p. 232.

great degree on estimates of what the conditions will be in 15 or even 20 years. As a matter of fact, by the time these 20-year estimates are 5 years old, the plant engineers will be asking for a new 20-year estimate, because some of the problems which arise in connection with plant extensions, office locations, etc., cannot be satisfactorily answered without such information.

Moreover, this is a practical proposition with the telephone company that affects, to the extent possibly of millions of dollars, the ultimate plans for its growth and extension. In Chicago, for instance, to maintain adequate service requires, and, apparently, will continue to require, $10 000 000 or $15 000 000 per year. The same, in proportion, is true of Detroit, Mich., New York, N. Y., and of all large cities; for this reason the telephone companies are directly and intensely interested in knowing the future distribution and character of population. It is vital to them to know where to locate operating centers, how big to make the conduit runs, and what size of cables to plan to use. This makes it important to know, for instance, whether the height of buildings in a certain area is to be limited to 10 stories, to 20 stories, or higher.

In connection with this much discussed question of the proper height of buildings, some of the conflicting opinions and the reasons therefor are amusing. For instance, facts are presented to show that very tall buildings are uneconomical, and then it is argued that those who are allowed to build them are thus given a commercial and economical advantage. It seems probable that if too many high buildings are erected in any area the public would object and decide to locate its offices in other parts of the city where high buildings do not exist. As a matter of fact, however, even in New York and Chicago a limited portion of the total area has been built up with very high buildings. An aeroplane photograph of Lower Manhattan or of the Chicago Loop District will illustrate this truth. Less than 3% of the area of the Chicago Loop is built up to 16 stories or more and yet the privilege of building as high as that has existed for many years. The average height in the Chicago Loop is less than 8 stories and regardless of legal permission a great deal of that area, as in Lower Manhattan, will never be built up to more than 5 or 10 stories.

However, if the height of buildings is to be legally restricted by zoning ordinance, or otherwise, the telephone company wishes to know the facts as soon as possible. It is much interested in the probable outcome of these zoning regulations in their effect either on the distribution of population, the height of office buildings, apartment buildings, or on the number of families that eventually will be allowed or can be expected in various sections of the city.

E. P. Goodrich,* M. Am. Soc. C. E.—The thanks of the membership for real contributions on the subject of City Planning are due those who prepared these papers, each of which stands out in a different category. Any engineer who goes before a Court in a particular case can probably turn to these papers and say, "This man, an expert on that particular subject, has said thus and so". For example, the late Professor Whipple discussed† a subject which is of the utmost importance, that is, health with relationship to zoning, and his paper

* Cons. Engr., New York, N. Y.

† *Proceedings*, Am. Soc. C. E., February, 1925, Papers and Discussions, p. 154.

unquestionably will be of value in its several aspects. There are, however, one or two items regarding which Professor Whipple is somewhat in error, as when he speaks about three-quarters of the ground area of the city being devoted to private use.

Professor Whipple also mentioned the desirability of establishing a relationship between size of buildings and area of streets on a scientific basis, which was also discussed by Mr. Crane.* That is something in which engineers particularly should have a part. In the past four or five years, the authorities of certain cities have undertaken to make this determination. In connection with the Plan of New York, very extensive investigations as to this relationship are being made.

It would be of great value if each one who can do so would make detailed traffic studies to determine the average length of travel of vehicles, the distribution of variation in length of trip, the average length of the walk of the man going from his place of sleeping, for example, to his office, or from his subway station to his office. These factors determine the density of street use and the widths of sidewalks and pavements; they re-act to determine in a definite way the proper height which should be placed on buildings. Fire prevention should also be an important factor in any logical study of building height limitation, but the economic factor is really the determining one, as it is in reference to all other phases of zoning. The psychological basis for the sentiment in favor of zoning, if it be analyzed, is doubtless an economic one. The man who owns a single family house wants it protected. He does not care subconsciously so much about the health of his children as he does about the value of his property. He gets out into the open for the benefit of his children and, thereafter, he wants the value of his property considered. The Courts, however, will not take into account the economic foundation that is the real basis of the desire. They will generally sustain reasonable restrictions, on health and on moral grounds, but, in the background, there is always this economic problem.

Mr. Crane refers to the height of buildings in Chicago, Ill., and the studies made by Mr. George C. Nimmons. A corner lot valued at $1 000 000 was used in this investigation, with building heights varying from 5 to 30 stories. He found the net return on the investment to be as follows:

Stories.	Percentage.	Stories.	Percentage.
5	4.36	20	7
10	6	25	6.72
15	6.82	30	5.65

There is, therefore, a clear-cut maximum rate of return on the investment, on the assumption that the land value is placed at $1 000 000, and that building heights vary. Normally, the land value for a building of that kind is equal to the value of the building itself. Unless that is the case, the depreciation in the building will increase faster than the enhancement in the value of the land, so that there is a gross loss to the owner. Now, if Mr. Nimmons'

* *Proceedings*, Am. Soc. C. E., February, 1925, Papers and Discussions, p. 194.

figures are revised to take into account land values which are equivalent to building values, the results are as follows:

Stories.	Percentage of Profit.	Stories.	Percentage of Loss.
5	3.6	20	0.2
10	1.2	25	1.1
15	0.04	30	1.7

In other words, wherever the proper economic relationship exists, high buildings are not a good investment, and it is always a surprise (as Mr. Bartholomew* has pointed out) that insurance companies and bankers will loan on such types of buildings. It is well known that normally they are erected either as an advertisement or as a monument.

The point brought out by Mr. Crane regarding 20-story buildings and 4-story buildings adjacent to the Loop District in Chicago, Ill., needs a little amplification. Mr. Crane states that where there are a certain number of 20-story buildings, with a limitation of the street use as to the pedestrians, in that same vicinity there will be a corresponding number of 4-story buildings. In consequence, the average of about 8 or 10 stories will be maintained for reasons of broad economics, because the community as a whole barely gets a return on its investment, although a few individuals net an extra return. Land values throughout the Loop District are practically uniform, and it is a question as to what building height will just support the value without starting a vicious circle of inflation.

Mr. Bartholomew brings out an interesting fact relative to zoning, namely, that area regulations have not been much discussed. The reason is that those who draw the regulations have not made them stringent enough. The case of an Omaha church is, possibly, the only one in which an area regulation has been brought into Court. In that case, the original regulation was upset because the church was in a "residence" district, and the restrictions were too rigid in the Court's view for that particular structure.

The important relationship between land subdivision and zoning was also stressed by Mr. Bartholomew. In several places in the Middle West, city planning commissions have secured the simultaneous promulgation of the zoning ordinance and of rules and regulations for platting. Furthermore, they have either formulated new building codes at the same time with the zoning ordinance, or provided, at least, that the zoning ordinance should be fully co-ordinated with the building code and, similarly, with the housing code.

The statement by Messrs. Sheridan and Hoffman† that streets cannot be legally widened under building line zoning should be emphasized. Nevertheless, it is an excellent means of securing indirectly what is desired. Sometimes it can be accomplished, but only rarely. Building line establishment, therefore, has been effected almost always by eminent domain.

As to Mr. Whitten's paper on "Housing Density Regulation"‡ it is felt that the stipulation of a number of families per acre, or a number of square

* *Proceedings*, Am. Soc. C. E., February, 1925, Papers and Discussions, p. 172.
† *Loc. cit.*, p. 214.
‡ *Loc. cit.*, p. 207.

feet per family, is unnecessary, is simply an encumbrance on the ordinance, and is difficult of administration.

Provided there are yards of proper size (of the types outlined by Mr. Bartholomew), plus a building height limitation and a proper use differentiation, sufficient data are at hand to limit automatically the families per acre, and thereby to distribute the congestion. With adequate sunlight planning, with proper sizes of yards and courts and widths of streets, and the control of room densities of population, a fundamentally sound basis is established for scientific zoning and building construction.

The paper presented by Mr. Knowles* is so good that it is difficult to discuss it. Most Boards of Appeals have no engineer member. Generally, either an engineer or an architect is stipulated, and because the architects are usually more numerous (if not more powerful) than the engineers, the architects get the appointment.

It seems to the speaker that both Pittsburgh, Pa., and Rochester, N. Y., have had a large number of appeals as compared with other cities, and a correspondingly large number of appeals granted, and if it were not for the personality of the individual members of the Boards the situation would be open to grave question.

This brings out the fact finally that the whole question is one that may be called "human engineering." Perhaps it is not yet possible to say that yards and courts should be of explicit sizes, due to sunlight and the reacting effect of sunlight on health (tuberculosis, rickets, and other diseases) as outlined by the late Professor Whipple, but the speaker believes that by analyzing the psychology of city planning it will be clear that zoning is an engineering problem for human engineers.

---

* *Proceedings*, Am. Soc. C. E., February, 1925, Papers and Discussions, p. 178.

THE INFLUENCE OF ZONING ON THE DESIGN OF PUBLIC UTILITIES

A SYMPOSIUM

Discussion*

BY MESSRS. MORRIS KNOWLES, ALBERT P. ALLEN, E. P. GOODRICH, AND WILLIAM T. LYLE

MORRIS KNOWLES,† M. AM. SOC. C. E.—The speaker is struck with Mr. Brewer's statement‡ that "any zoning ordinance should require every new subdivision to have at least a 2-acre playground for every 160 acres subdivided."

The question arises, is it wise to try to accomplish this through the police power of the zoning ordinance? It suggests at once the type of regulation so frequently attempted under zoning—an excellent idea in itself—but a problem which should come under the broader subject of city planning, leaving under zoning only the things that properly belong there.

The speaker has been much interested also in the discussion of the relation of zoning to telephone problems by Mr. Allen.§ The author holds that the rate-payer is the person who suffers. This fact is important in all public utility problems. It is the same with regard to the paving of streets by street railway companies; or the paying of toll charges for water lines or gas lines crossing bridges; or the pole taxes for which the electric light companies are assessed; or mileage taxes for gas companies having pipe lines in the streets. Some of the charges of this kind have aimed to provide municipal revenues, rather than to pay the cost of municipal inspection; from that point of view they are not economically sound. In attempting such assessments, the people have forgotten that it is the rate-payer who pays.

This is also true in the telephone business, illustrating an important consideration in zoning, namely, that in all attempts to be theoretically perfect, practical limitations must not be forgotten. For example, in a desire to zone thoroughly for residence districts, care should be taken not to impose such conditions as will make all the residents pay additionally for this common and ordinary convenience. It seems as if there is no good reason why the buildings of a telephone system, when needed in a residence district, should not be considered in somewhat the same classification as a church, a school, a hospital, or

* This discussion (on the Symposium on Zoning presented at the meeting of the City Planning Division on October 25, 1924, and published in February, 1925, *Proceedings*) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Pres. and Chf. Engr., Morris Knowles, Inc., Pittsburgh, Pa.

‡ *Proceedings*, Am. Soc. C. E., February, 1925, Papers and Discussions, p. 244.

§ *Loc. cit.*, p. 232.

possibly a fire station. These are not residences, certainly; but they are buildings which naturally are placed in localities where people live, because they are a convenience, and because it is an advantage to have them in residential neighborhoods. Therefore, it is worth while to make a definite statement in the zoning ordinance with respect to this subject. The public who pays the bills is interested.

Frequently, one is met with the suggestion—as perhaps appears partly in Mr. Allen's paper—that if there is adequate legislation, by the aid of a thorough enabling act, and an ordinance that is sufficiently comprehensive, based thereon, with a Board of Appeals, then there is available all the machinery by which proper justice can be given when the time comes. However, just a little reflection will indicate that this is hardly practicable; although the speaker will confess that at first this was his own point of view.

In most situations, a telephone company must scheme to secure sites, far in advance, for the future location of central stations. If in that case it announces what it wishes to do by appearance before a Board of Appeals to secure permission, the holders of property in that immediate vicinity will advance the price, and it will become increasingly difficult to secure property at all, or to secure it at a reasonable figure. In the end, the public will pay, and hence it is apparent that some other means must be adopted.

For a long time the speaker was somewhat puzzled as to why there should be any distinction between a telephone system, and an electric light, a gas, or a water supply system. In the telephone system one pair of wires goes from the central station to each subscriber, that is, one wire goes and another comes back, while with a gas or water supply there is only one main along the street with taps for the house services. The complicity of the telephone arrangement naturally limits the area that may be economically controlled from any one central office; and unless the sub-stations can be placed at proper locations, as determined by design, the weight of copper multiplies rapidly and the cost becomes excessive, especially if the subscribers are a long distance away.

In Pennsylvania, some time ago, a committee was appointed to see what regulations appeared reasonable. This committee had an opportunity to listen to the explanation of the officers and engineers of the telephone company in Pennsylvania, and the criticisms and discussions of those who represesented the municipalities.

The report submitted was in part as follows:

"The Committee is convinced, after listening to and considering the able presentation of the problem involved in locating 'Telephone Central offices' close to wire centers, that a real necessity may exist for such structures in 'Residence Districts' * * *.

"The Committee is of the opinion that 'Residence Districts' should be relatively small, and such that any part is not far removed from a 'Business District'. When such is the case, the need for 'Telephone Central Offices' in a 'Residence District' may be less acute than otherwise. In non-populous communities, where one central office will serve the entire telephone community for many years to come, the municipal authorities in their discretion may decide to omit a 'Telephone Central Office' from such list of permitted uses in 'Residence Districts', and under the head of 'Exceptions' such use may be granted, under appropriate restrictions. In such cases the Board of Appeals

should be given the power to make a variance in the case of a special application and allow a 'Telephone Central Office' in a 'Residence District' if, in its opinion, it will not be unreasonably injurious to the surrounding property.

"However, while the making of 'Zoning Maps' is progressing toward accuracy in definition, in view of the application of zoning as now practiced upon maps and to meet the border line cases between municipalities, the Committee is of the opinion that the clauses given under (2), (3) and (4) are proper to have inserted and so recommend."

Clause (2), to which reference is made, reads as follows:

"(2) That in the list of definitions, under the suitable section of a drafted zoning ordinance, there be inserted, in the proper alphabetical order, a definition of 'Telephone Central Office'.

"*Telephone Central Office.*—A building and its equipment, erected and used for the purpose of facilitating transmission and exchange of telephone messages between subscribers, and other business of the Telephone Company; but in a residential district not to include public business facilities, storage of materials, trucks or repair facilities, or housing of repair crews".

However, as was stated by Mr. Allen, possibly the Company should be permitted to erect buildings of that type, higher than the customary limit within the district, and that they should occupy a larger percentage of the lot. The general feeling of the Committee was that this is the type of regulation which should be controlled by the Board of Appeals, because the Company could first secure the land without publicity, and, therefore, at a reasonable price, but, after having secured it, and knowing that it was safe, it could go to the Board of Appeals and make request for any necessary changes in height or limitation of area. It would have to show good cause, and the Board of Appeals would have the right to impose certain conditions. Although an ordinance may not under most jurisdictions impose conditions which are unesthetic, or out of harmony with the neighborhood, the Board of Appeals may impose some conditions as to the character and details of construction, such as the percentage of window openings, the restriction that a door may not open on a certain street, or any provision that would overcome conditions that might be objectionable in a residence district. Such restrictions could not be imposed by the ordinance.

The report concludes with Clauses (3) and (4) previously mentioned:

"(3) That, where the municipal authorities deem it necessary, as above stated, there be listed under permissible uses in 'Residence Districts'—whether classified in one or more different districts—'Telephone Central Offices'.

"(4) That there be included under listed 'Powers of the Board of Appeads', preceded by the inclusive phrase, 'In appropriate cases the Board may cause to issue a permit'.

"In connection with the erection and use of a building in any Residence District for a Telephone Central Office, for such variation of height and area restrictions as may be necessary for the suitable use of the structure; * * * and not unreasonably injurious to the property of the district in which the structure is proposed to be located."

Such provisions in a zoning ordinance will permit all that is reasonably required. It will be possible to purchase the land in the desired locations without publicity and, therefore, at a reasonable cost; the Board of Appeals,

upon application, may grant such extensions to height and area as may seem desirable; and in doing so may impose such conditions as may be needed for the protection of the surrounding properties.

ALBERT P. ALLEN,* ESQ.—The transportation system in a city, and especially its system of fares, undoubtedly has, or can be made to have, a noticeable effect on the distribution of population. Incidentally, the proper distribution of population throughout the available residence districts surrounding an economic business district is one of the principal objects of zoning; and the tendency to-day in all large cities is to insist that the street car fare must be a flat or fixed fare, with universal transfers, allowing a passenger to ride from one side of the city to the other at no greater cost than is required to ride a few blocks. In the speaker's opinion this system is absolutely wrong from an economic standpoint, and also from the standpoint of the best interests of the public itself, as indicated by the results that it has on the growth and distribution of population and the congestion of traffic on street and transportation lines which follows such distribution.

If street car fares were put on a more economic basis, which really means that the rate of fare should recognize to some extent the mileage involved, the change in the usual practice will be a great factor in establishing the proper distribution of population around secondary business centers, in keeping down the transportation between such centers, and in confining it within such neighborhood areas through the influence of the proposed mileage differential in fares.

For instance, in Chicago, a housewife living near the city boundaries can go down to the Loop District to shop with no more cost for transportation than if she went half a mile to a neighborhood store. This fact tends to congest transportation lines to the Loop and creates the greatest confusion there. If, however, she could ride to a neighborhood center for 5 cents and have to pay 7 to 10 cents to go to the Loop, there would be, no doubt, a noticeable reduction in the crowding.

The speaker remembers one case where a 2-cent charge for a telephone message has cut in two the number of messages previously handled on a "free service" basis. It might not seem that anybody would hesitate or change his practice for a differential of 2 cents, but unquestionably he will. Some rate differential in connection with local transportation is desirable, not only for the reasons already discussed, but also because it allows the transportation system to retain the cheap, but profitable, short-haul business which to a large extent is entirely lost when the average fare has to become 7 or 8 cents instead of 5 cents; and this fact in itself makes necessary an increase in the average fare which is greater than it would be if such short-haul traffic could be retained. It is quite probable, therefore, that the placing of street car fares on a logical, economic basis by paying at least some attention to mileage, possibly through a reasonable charge for transfers, will help the zoning of large cities through an improved distribution of population.

* Commercial Engr., Illinois Bell Telephone Co., Chicago, Ill.

E. P. GOODRICH,* M. AM. SOC. C. E.—Practically every paper presented as a part of this Symposium brings out by inference the definite need of establishing a relationship between zoning and transportation.

As a previous discussion concerned the relationship between bulk and street traffic, so the same type of investigation must be made with reference to building bulk and the main transportation system, the street railway system, the busses, etc. Many of the items discussed by Mr. Bibbins† will be of interest in this regard, but so far they have not been properly correlated. That is the speaker's only criticism of his paper; it is regretted he has not gone farther to produce something more constructive.

In making that kind of a study, there are two or three methods which possibly should be followed. In Mr. Phillips' paper,‡ the need of analyzing the zones of origin and destination, and of relating them, was suggested. That must also be done with reference to pedestrian travel, to vehicular travel, and to rapid transit transportation. As yet there is no large public consensus of opinion, or system of facts, with reference to these things. The speaker gladly gives, for whatever they are worth, the results of certain investigations he has made, in the hope that other engineers will make similar investigations, and publish the results.

Normally, the pedestrian walks as short a distance as he can, except during the noon hour. Then, there is a sauntering, recreational condition. Now, it is hardly to be expected that sidewalks will ever be designed for recreational purposes (except possibly at places like Atlantic City, N. J.); some other place should be provided (on the tops of buildings, perhaps,), the sidewalk space, the street area, being set aside for movement for business purposes. The average distance which a person walks during other than the noon hour is about three blocks, and the rate is about 2.8 miles per hour. He goes more rapidly to his office in the morning than home at night. The floor area occupied by the average office is about 100 sq. ft. per person, including space for transients as well as for the permanent residents. One large building in New York, which may be considered as typical, because it has been checked in comparison with buildings in quite a number of other cities, discharges 1 700 people per hour. It occupies an entire block, and is 1 400 ft. in circumference.

Turning now to automobiles: One family in an average general residential district (including apartment houses) has the equivalent of about one vehicle per day coming to it and departing. This condition has been found to exist in all parts of the country and over large areas of varying economic character, it is an average figure. Obviously, in the districts of lower economic levels, the rate is less, but, in some other districts, it is correspondingly higher. Rural stores, or outlying stores, one story high, create and receive the equivalent of 1 vehicle per 5 running feet of frontage. The relationship which exists between the residential area and the store area is evolved as follows: On the average, one family is found per lot, 40 by 100 ft.; in outlying districts,

* Cons. Engr., New York, N. Y.

† *Proceedings*, Am. Soc. C. E., February, 1925, Papers and Discussions, p. 247.

‡ *Loc. cit.*, p. 224.

the lots are larger. Toward the center, the apartment houses reduce the average area of land per family, but the figure given applies very generally over large areas of most cities. A store with a 25-ft. front will normally serve about 100 persons. The density of use of industrial areas varies considerably, but about 4 000 sq. ft. per vehicle per day has proved a fair average.

Now, when the proper zoning ordinance has been prepared for a community, so that the relative areas assigned to residence, business, and industry, are approximately balanced, so that there is an excessive industrial area with no available residences, or so that, as in some instances actually happens, a whole section of the community has been set aside for business and never can be used fully for business purposes, with a depreciating effect on the vacant properties and a most depressing effect on the business itself—when this proper relation has been deduced, then one can build up a street system, a drainage system, a telephone system, a recreational system, and all the other things discussed, to correspond with these particular types of development.

To-day most zoning requirements are too loose. They do not begin to make restrictions to the point which the evidence demonstrates that the needs already demand. Generally speaking, building heights are excessive. In a few communities they are reasonable in comparison with street widths, but these cases are rare; most zoning ordinances are useless as far as setting limits within which one can design a street system and other facilities. The actual distribution must also be taken into consideration. There is only one ordinance among the one hundred with which the speaker is familiar under which there would be a limiting of the actual total population. The remainder are just starts in the right direction. To do anything with zoning that will affect the future with respect to transportation, for example, requires an entire change of attitude.

Respecting the problem of transportation and building height, assume a central distributing point, for instance for coal for the community, or a central market distributing food products, or a central district to which crowds come every day, as an office building district. If there is a low height limit—and, assume for this purpose, that the height is uniformly built up throughout the community—it is plain that the area required will be much larger than if there is a higher limit. Now, to supply coal from the central point requires an average haul which is considerably longer in the case of the low limit than in the case of the high limit; also, a longer telephone cable system, and a much larger water supply system. As to the streets, they will receive much more traffic, so that around the center a greater congestion will exist with the lower than with the higher height limit.

Obviously, judged solely on this argument, it is advisable to make all buildings just as high and just as compact as possible. That line of reasoning results in an extreme condition against which zoning is largely aimed. Somewhere there lies an economic balance toward which engineers should strive. The telephone companies have done more work than any other group on this problem. For example, the cost of displacing a telephone central from its natural economic location has been estimated for one specific case in an Eastern city. A diagram was prepared by the telephone engineers, giving

the actual increase in capital involved by moving one, two, three, or four blocks from the natural center. If the telephone central station (serving 7 500 subscribers in this particular instance) were displaced ½ mile, it would involve a capital charge, which, at 6% interest, spread over the subscribers, would result in an increase of rates of 76½ cents per month. If there were a displacement of ¼ mile, there would be an increased charge of 26.6 cents per month which would normally be applied to the rates. Therefore, the question for the subscribers and for any community to determine is whether the benefits which will accrue from zoning are greater or less to the individuals as a whole than the extra costs incurred.

The same is true of street traffic. Is the congestion (which is costly) greater or less in cost than the remedy? The old adage that "the cure is often worse than the disease," often applies economically.

The paper by Mr. Brewer* is particularly interesting from an engineering standpoint by furnishing data with reference to the use of playgrounds by individuals of different ages.

The subject of zoning is one on which engineers can well afford to spend much more time than they have in the past. It is one which needs the engineering method of approach, especially that of the economic engineer. The engineer is not merely a man who can design, but a man who designs for the least cost. The whole structure of modern urban life is so bound up with the economic features, with the cost (and zoning is the one thing to-day which makes an appeal fundamentally on an economic basis, although it is based also on health and safety and similar things), that the engineer from his economic point of view can well afford to delve deeply into it.

WILLIAM T. LYLE,† ASSOC. M. AM. SOC. C. E. (by letter).‡—More than two thousand years ago, using the deductive method of research, Euclid exhausted the subject of Geometry and Aristotle, the subject of Logic. Much of the science of City Planning also can be reasoned deductively; and much remains to be learned from experience. Deduction is useful: induction is absolutely necessary. Zoning is a new art the successful application of which depends on a careful engineering study of conditions as they exist. No two cities require the same treatment. Engineers are not yet equipped with codified principles and rules concerning the conformation of the districts of a zoned city, sufficient to govern procedure. Each zoning plan must be developed as a case by itself.

The subject under discussion is well stated by Mr. Brewer.* Zoning influences each and all of the utilities mentioned, but each utility cannot be considered by itself in its influence on zoning; it must be studied in its relationship to each and all of the others. Thus, for example, the location of park reservations may be determined by that of storage and impounding reservoirs, the park protecting the water supply from contamination and the reservoir pro-

* *Proceedings*, Am. Soc. C. E., February, 1925, Papers and Discussions, p. 242.

† Prof. of Civ. Eng., Washington and Lee Univ., Lexington, Va.

‡ Received by the Secretary, October 15, 1924.

viding a scenic feature in the park. So, also, the street and transportation systems are intimately related; also boulevards and drainage.

The writer does not feel qualified to present the principles of zoning in its influence on recreation facilities in a systematic form. He will be content merely to venture a few statements from his own experience.

Connected parks imply a connected and continuous high-class or first residential district. The parks are connected by boulevards or parkways which provide valuable residential frontages. Zoning thus established is not subject in any considerable degree to controversy and, therefore, possesses great stability.

Connected parks also control to a lesser degree the second residential district which borders the first. This is due not only to the proximity of the first district, but also to the recreation facilities near-by.

A belt line, or surrounding boulevard or parkway, implies a belt of first and second residential districts. These districts are thus systematically located, which of course is a great advantage. In some measure the city thus acquires geometrical conformation. As boulevards and parkways frequently lie along watercourses, the improvement may be combined with that of storm-water drainage. Brooks and creeks outside the city limits may be utilized in the location of a surrounding boulevard or parkway, the boulevard or parkway cutting across the intervening ridge from one watercourse which it has followed to another. Such watercourses are commonly edged with trees. The trees and water provide the architectural basis of development. These channels often are fouled by sewage and refuse, and, being useless for other purposes, can be secured at small cost. They may be dredged and regulated, and provided with drives and other architectural features. The removal of the nuisance is incidental to the improvement.

The artifice of building park "border mounds" is open to criticism in that they shut off park views from the surrounding streets and houses. The border mound properly functions in providing seclusion in the park by interposing a barrier of trees and shrubbery between the interior of the park and the ugly buildings surrounding it. Where the park is bounded by buildings of the first residential district the need of such a barrier is not so imperative. Seclusion can be had in sufficient measure by judicious planting, aided by a low mound set back from the street line. By proper architectural treatment a reasonable seclusion can be had for the park while at the same time permitting short views into the park from the outside.

Radial streets and boulevards reaching out to the surrounding boulevard or parkway imply expanding second residential districts developing outwardly into the first. Streamers from the business district will extend for some distance toward the surrounding boulevard or parkway. Between these streamers near the heart of the city, will be found the light industrial district, and farther out the second residential district. Separation is natural and easy.

Promising areas can be encouraged to develop as they should by the establishment of parks and playgrounds, with boulevards leading centrally toward the commercial district and circumferentially from one park to another. The writer has in mind a most promising area on the west side of Houston, Tex.,

along the Buffalo Bayou, an area the value of which has not yet been fully appreciated. This might be converted into a first residential district with a parkway as a basis of development. The parkway should include the stream and be provided with drives on both sides. This part of the town is now one of the negro settlements; the natural asset of the stream is wholly lost.

Playgrounds, public gardens, and city squares are needed in the second residential districts between the radial streamers which extend outwardly from the business district. These are the gores where blighting is likely to spread with great rapidity. It can be checked in some degree by the establishment of recreation facilities.

Improper development of the heavy industrial district can be checked by judicious park layouts, saving the residential territory from serious blighting and keeping the heavy industries within due bounds near the railroads, where they properly belong. The railroads should be grouped and centralized, and not allowed to destroy the value of land better suited to other uses. At the same time, every possible encouragement should be given to the railroads and the heavy industries on which the prosperity of the city depends. The interests of recreation facilities are not antagonistic to those of business and industry. There is no better method of aiding business and industry than by improving the living conditions of those on whom the success of business and industry depend.

MEMOIRS OF DECEASED MEMBERS

Note.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

DANIEL BONTECOU, M. Am. Soc. C. E.*

Died July 14, 1924.

Daniel Bontecou, the son of William Ely and Caroline C. (Thayer) Bontecou, was born on September 14, 1851, at Springfield, Mass., the home of his grandfather, Daniel Bontecou. His early boyhood was spent in Weymouth, Mass., where he attended the English High School of Boston. Later, the family moved to New York, N. Y., and at the age of 16 years Mr. Bontecou, through his own efforts, entered the College of the City of New York, from which he was graduated in 1871, second in his class, with the degree of Bachelor of Science.

He began his professional career in New York, in the Department of Public Works, where he was employed from 1871 to 1873. He was then appointed Assistant Engineer with the New York Central Railroad Company and assigned to duty at Fonda, N. Y., where he was in charge of 28 miles of the four-tracking of that System, and also of work at the 59th Street Terminal in New York. On the completion of this work, in 1874, he became Assistant Engineer on the location survey for the railway line between Albany, N. Y., and the Hoosac Tunnel.

In 1876, Mr. Bontecou was appointed Assistant to the late Elnathan Sweet, M. Am. Soc. C. E., then State Engineer of New York, on the New York State Canals, in which position he continued until 1878, when he returned to the employ of the City of New York, and was in charge of the construction of the Madison Avenue Bridge and of plans for the Washington Bridge and other bridges across the Harlem River. His service with the City of New York continued until 1881, at which time he removed to Kansas City, Mo., and formed an engineering partnership with the late William B. Knight, M. Am. Soc. C. E., under the firm name of Knight and Bontecou, to carry on a general civil engineering practice.

In 1883, Mr. Bontecou was appointed Chief Engineer of the Kansas City Belt Railway Company, in which position he continued until 1886, when he became Chief Engineer of the Grand Avenue Railway Company, of Kansas City. During 1889, he was called to Washington, D. C., and appointed Chief Engineer for the Capital Traction Company of Washington, on the construction of the cable-operated street car lines and power plant of that city, where he remained until their completion and operation in 1890. Mr. Bontecou then became Chief Engineer of the Kansas City, Fort Scott, and Memphis Railroad and Associated Companies, including the Kansas City Belt Railway, which position he retained until 1901. From 1901 to 1918, he was engaged in private

* Memoir prepared by Robert Ridgway, President, Am. Soc. C. E., and Eugene G. Haines, M. Am. Soc. C. E.

practice in Kansas City as Consulting Engineer, and in this capacity he was retained in connection with numerous projects of many different types and clients, including the following: Viaduct plans for the Metropolitan Street Railway Company; the Kansas City Lighting Company; the Central Avenue Power House; the Kansas City Stock Yards; the design and construction of Kansas River bridges, and water supply; the Capital Traction Company of Washington, Pennsylvania Avenue Cable Lines; the Kansas City Cement Company and the Hawkeye Cement Company; the Kansas City and Western Railway Company on valuation and line improvements; steel and concrete buildings, power installations, etc., for the United Zinc and Chemical Company; reports to the Fidelity Trust Company on the Kansas City and Missouri Interurban Railway including plans and expenditures; the Oklahoma, Kansas and Missouri and Okmulgee Northern Railways; the Kansas City and Missouri River Navigation Company, traffic studies and warehouses; the St. Louis and Santa Fé Railroad on revision of valuation of the Arkansas Lines; report to creditor's banks on the cost of building the Galveston Causeway; and reports to railway companies and others on the cost and details of reconstructing fifteen bridges after the Kaw River floods in 1903.

Mr. Bontecou was a member of the Advisory Board of Engineers on the matter of industrial preparedness, representing the State of Missouri and working under the Naval Consulting Board in 1916. He also served the Government in other ways, reporting, in 1918, to the U. S. Shipping Board on "The Availability of Inland Waterways to Relieve Railroad Congestion", and to the Inland Waterways Commission on "The Traffic of the Mississippi River above St. Louis"; he also served as District Engineer with the Housing Corporation in charge of the proposed developments at Perth Amboy, N. J.; Port Penn, Del.; and Sheffield, Florence, and Tuscaloosa, Ala. He was a member of the Engineers Club of New York.

It is not easy to define the personal characteristics which, taken in the aggregate, mark a man as different from many others, for they represent only the outward reflection of the rays from many qualities, often elusive and not pronounced, but where many good qualities are combined in one man, he is known as a man of "fine character", and such was Daniel Bontecou. A composite picture, drawn by quotations from letters received since his death, reveals him as: "A very quiet man" of "pleasant manner", and "always a perfect gentleman"; "one you could go to for advice at any time and always meet with a kindly word and good advice". He was "an able engineer", "studious", "one of the most thorough men I have ever met", "a clear thinker", and, "one who loved to work out intricate details". He was a "close observer" and "quickly came to know the worth of the men under him and their ability". He "inspired confidence", and those with whom he became associated "learned to know and love the man" as one "inspired by the highest motives" and of "the very highest ideals and character", and they placed "great faith in him as an engineer". He was "a most devoted son", "a loyal citizen", and "a warm personal friend". What a tribute this is to the memory of a man; and yet it has all been said, in one brief and sweetly beautiful sentence, by one who knew him best and who kept pace beside him for thirty-nine years as his wife: "He

died, as he had lived, bravely and patiently; and he leaves a wonderful memory for his children, and for me."

Mr. Bontecou was married in 1885, at Falmouth, Mass., to Nathalie Holdrege, of Irvington, N. Y. His death occurred at Mamaroneck, N. Y., where he had resided since his return to the East, and he is survived by his wife and five children, Daniel Jr., Russell, Frederick H., Helen, Nathalie Bontecou Crocker, and by six grandchildren.

Mr. Bontecou was elected a Member of the American Society of Civil Engineers on November 5, 1879. He served as Director from 1896 to 1898, and as Vice-President in 1915 and 1916.

JOHN GRIFFITHS BROWN, M. Am. Soc. C. E.*

Died October 22, 1917.

John Griffiths Brown, the son of James and Mary A. Kerr Brown, was born on June 1, 1873, at Philadelphia, Pa.

He was educated in the public schools, and was apprenticed in architectural wood-carving in his father's shop. His contact in his work with architects and engineers developed in him the ambition to become an engineer, and on finishing his apprenticeship he secured employment with the United States Engineer Corps on survey work along the Delaware River, and, subsequently, a position in a surveying corps on the New York subways.

In 1901, Mr. Brown became a Cement Inspector on the Atlantic Avenue Improvement in Brooklyn, N. Y., which position he held until, in 1903, he was appointed Inspector of Cement and Concrete Material for the Philadelphia Rapid Transit Company, when it entered on construction of its Market Street Subway and Elevated Line.

In this position he pursued his study of structural design as well as of the technique of concrete making, and, in 1904, became General Manager of the Unit Concrete-Steel Frame Company of Philadelphia, engaged in the construction of buildings of pre-cast reinforced units.

In 1906, Mr. Brown established himself in business as a Designer and Contracting Builder, specializing in concrete industrial buildings—mills and factories—his service covering the whole range of the study of plant requirements, the production engineer's field, surveys of sites, designs of buildings, and construction and equipment.

Among the many operations prosecuted by him are the Fayette R. Plumb Plant of sixteen buildings, St. Louis, Mo.; the Savage Arms Plant, Utica, N. Y.; the Grellet Collins Building, Hale and Kilburn Factory, Electric Storage Battery Company Building, and William F. Read Sons Building, Philadelphia; the American Malleable Company's Plant, Buffalo, N. Y.; the Ingersoll Watch Company, Trenton, N. J.; the Mack-Saur Motor Company, Plainfield, N. J.; the Wall Rope Works, Beverly, N. J.; and the Jeansville Iron Works, Hazleton, Pa.

* Memoir prepared by Henry H. Quimby, M. Am. Soc. C. E.

Mr. Brown's business policy was a far-sighted one—to obtain clients through notable devotion to their interests. His contracts for construction as well as for design were on a percentage basis, but he studied and strove to economize in his client's interest. He built up an efficient organization of specialists, and systematically provided technical instruction for his employees by professional teachers and experienced practical men, furnishing the books and instruments as well as the teachers, the instruction covering practical points of work, design of structures, and construction equipment, inspection, architecture, and psychology. He called for intelligent as well as faithful devotion to work, believing in the necessity of thorough vocational training.

Mr. Brown held and presided at periodical organization meetings, attended by the heads of departments, foremen, engineers, draftsmen, office employees, and apprentices, at which all branches of the business were freely discussed, differences were ironed out, and co-operation was promoted.

He was of a genial, kindly disposition, with a host of friends, energetic in technical society activities as well as in his professional work; he placed high value on character in his associates and on uprightness in his employees, investigating thoroughly the habits of an applicant before employing him.

Mr. Brown was a member of the American Society for Testing Materials, and of the Engineers' Club and the Manufacturers' Club of Philadelphia.

He was married to Margaret B. Taylor, on November 2, 1901, who survived him for several years. They had no children.

Mr. Brown was elected an Associate of the American Society of Civil Engineers on April 3, 1906, and a Member on September 3, 1913.

ANDREW ARNOLD COHILL, M. Am. Soc. C. E.*

Died July 2, 1924.

Andrew A. Cohill, the son of Andrew A. and Anastatia Egan Cohill, was born in Williamsport, Pa., on December 9, 1869. His father, who was a Civil Engineer, built most of the early bridge work of the Pennsylvania Railroad and was in charge of the old Pennsylvania Canal, which went out of existence in 1889, at the time of the Johnstown flood.

Mr. Cohill received his early education in the public schools of Williamsport, and, later, attended a private school where he studied engineering. On the completion of his course his father obtained for him a position with the Engineer Corps of the Pennsylvania Canal which was engaged in setting 1 000-yd. property monuments. He afterward served on the original survey of the Watsontown and Shickshinny Railroad from Watsontown to Shickshinny, Pa., which was known as the Watsontown and Western Railway. After spending considerable time in survey work, Mr. Cohill accepted a position at Altoona, Pa., in the Engineering Department of the Pennsylvania Railroad, Pittsburgh Division.

In 1890, many of the Pennsylvania Railroad officials transferred their services to a more recently established railroad—the Norfolk and Western.

* Memoir prepared by Hunley Abbott, M. Am. Soc. C. E.

Mr. Cohill, who went with them, was made Assistant Supervisor at Pulaski, Va., and shortly afterward was given the position of Supervisor of the New River Division, between Radford, Va., and Bluefield, W. Va., with headquarters at Oakvale, W. Va. At this time the double tracking on this Division had begun, and it was noted by officials of the Norfolk and Western Railway Company that they were assigning their youngest Supervisor, not only in point of service, but in age, to the most important Division. Mr. Cohill executed the work of this Division ably throughout the entire period of double tracking.

Mr. Cohill then accepted the position of Engineer for Clark and Westbrook, Contractors, who were building a four-track road on the New York, New Haven and Hartford Railroad, at Mount Vernon, N. Y. This was heavy construction work, the length of the section extending from Woodlawn to New Rochelle, N. Y., a distance of approximately 5½ miles, and the tracks, which originally passed through Mount Vernon at street level, were depressed at some points as much as 30 ft. below the original grade.

Mr. Cohill later became associated with the Schoen Pressed Steel Car Company, at Pittsburgh, Pa., at its old Alleghany Plant. This was at the beginning of the pressed steel car industry. He was made Superintendent, but soon resigned to become Superintendent of the Keystone Bridge Works at the time the American Bridge Company was organized in 1900.

In 1902, Mr. Cohill was given the engineering as well as the construction work on the new furnaces for the National Tube Company at McKeesport, Pa. During a period of two years he built a new pumping plant and placed a new foundation under the furnace, keeping it in operation at the same time.

From 1904 to 1906, he served as Superintendent and Engineer on construction work for the American Telephone and Telegraph Company, building an underground system between Boston, Mass., and Providence, R. I., Trenton and Camden, N. J., and Racine and Milwaukee, Wis. From 1906 to 1908, he held the position of Superintendent and Engineer for S. Pearson and Son on the construction of the Pennsylvania Railroad Tunnels under the East River, New York, N. Y., and, from 1908 to 1909, he studied and traveled abroad.

Mr. Cohill was connected with T. A. Gillespie and Company from 1909 to 1912, serving one year on the construction of retaining walls along the Monongahela River, at Pittsburgh, Pa., and two years as General Superintendent on Contract No. 68 of the Catskill Aqueduct, including the construction of seven steel siphons, from Peekskill to Yonkers, N. Y. From 1912 to 1914, he held the position of General Superintendent for James Stewart and Company on the New York State Barge Canal Contracts 5-A, 59, and 72-A, and, in 1914, he joined the staff of P. McGovern and Company, with which Company he remained until 1920. He served for two years as Superintendent and Chief Engineer on the construction of the State Barge Canal Contract 71-A, and on the construction of Fort Point Channel, Boston, and for two years as Chief Engineer on the construction of Route 61 of the New York City Subways, comprising elevated, subway, compressed air, and hard rock tunnel work under the East River at Blackwells Island, and also on the construction of cylinder foundations for the Quartermaster's Terminal, at Boston.

In May, 1920, Mr. Cohill left the employ of P. McGovern and Company, and shortly afterward became associated with large financial interests in the preparation of a bid on a vehicular tunnel, at New York. This work engaged his attention for considerably more than a year. His estimate was carried out in the finest detail, but at the last minute his employers decided not to submit his bid. If they had done so, his estimate would have been slightly under those of all other bidders on this work and, undoubtedly, would have secured the contract.

During the last two years of Mr. Cohill's life, he served as General Manager for Abbott, Merkt and Company, Industrial Engineers, of New York.

He was not only an experienced Engineer, but particularly capable in handling construction work and directing men in the field. He quickly gained their confidence, and thereafter commanded them with all the precision of a military organization. Few men of his generation knew more than he concerning the theory and practice of difficult tunnel construction. His work on the foundations of the Quartermaster's Terminal in Boston, during the World War, set a record for speed in this type of work that probably never will be equalled.

Mr. Cohill was endowed with a most lovable personality, being a good friend to all with whom he came in contact and generous almost to a fault. He is survived by Mrs. Mercedes Good Cohill, to whom he was married at McKeesport, Pa., in October, 1903.

Mr. Cohill was elected a Member of the American Society of Civil Engineers on January 13, 1919.

NELSON PETER LEWIS, M. Am. Soc. C. E.*

DIED MARCH 30, 1924.

Nelson Peter Lewis, the son of John Neher and Christina Jane (Nelson) Lewis, was born at Red Hook, Dutchess County, N. Y., on February 1, 1856. He received the degree of A. B. from St. Stephen's College, Annandale, N. Y., in 1875, and was graduated as a Civil Engineer from Rensselaer Polytechnic Institute in 1879. The honorary degree of LL.D. was conferred on him by St. Stephen's College in 1911.

Mr. Lewis achieved an enviable reputation both in the United States and abroad as a municipal engineer and city planning expert, and as such his services were invaluable to the City of New York. He began his engineering practice, however, in railroad location, first with the Denver and South Park and the Denver and Rio Grande Railroads in Colorado, in 1881, and from 1882 to 1884, with the Vicksburg, Shreveport and Pacific Railroad in Louisiana, following which, for three months, he was Supervisor of Track with the Delaware and Hudson Railroad.

On July 1, 1884, Mr. Lewis joined the Engineering Staff of the Department of City Works of Brooklyn, N. Y., as Assistant Engineer on an extension

* Memoir prepared by the following Committee: Arthur S. Tuttle, *Chairman*, Richard S. Buck, and J. Waldo Smith, Members, Am. Soc. C. E., and George H. Pegram and George S. Webster, Past-Presidents, Am. Soc. C. E.

of the Water-Works System. From this time, with the exception of the period from July 1, 1886, to July, 1889, when he was with the Central Railroad of Georgia, in charge of railroad location and construction and of the planning of terminal and water-front improvements in Alabama and Georgia, he remained in continuous municipal service, first with the City of Brooklyn until the formation of Greater New York in 1898 and after that date with the Consolidated City.

In 1894, the late Alfred T. White, Commissioner of City Works of Brooklyn, appointed him Chief of the Bureau of Local Improvements, which step he has described, as follows:

"Alfred T. White was Commissioner of City Works of Brooklyn in 1894 and 1895. One of the Bureaus of the Department had to do with street improvements. Its head was a man whose previous occupation had been—I do not remember what—plumber or possibly an undertaker or some quite honest business, but it did not conspicuously fit him to administer a Bureau of the City Government which had to do exclusively with engineering work. Mr. White believed that an office of this kind should be administered by an engineer and he designated me as its head, with the idea that it should be placed upon an engineering basis; but he did more than this—he gave me a free hand in the re-organization of the office and in the selection of a staff of capable assistants."

In this position Mr. Lewis was able to build up an engineering staff that contained many men who have since been active in the various engineering organizations of the city.

For the four years beginning with 1898, he was Chief of the Brooklyn Bureau of Highways; it was during this period and under his leadership that the substitution of the present type of smooth pavement for the old system of cobblestone highways in that Borough, was undertaken.

Mayor Seth Low noticed the progress which was being accomplished by Mr. Lewis in Brooklyn and appointed him as the first Chief Engineer of the Board of Estimate and Apportionment of New York City in March, 1902. He held this office through successive administrations until his retirement in December, 1920. This position called for reports on all engineering projects brought before the Board of Estimate and Apportionment for approval, and demanded a combination of professional skill, judgment, integrity, and tact, with which Mr. Lewis was exceptionally well equipped. His activities were largely responsible for the orderly development of the plans under which the growth of the city was shaped, as well as for the successful functioning of the various agencies created for carrying on the vast engineering work. Undoubtedly, his judgment often saved the city vast sums of money.

Mr. Lewis served on a number of committees or commissions appointed by the Mayor or the Board of Estimate and Apportionment, such as the New York City Improvement Commission, the Commission to Investigate the Subject of Municipal Lighting Plants, the Heights of Buildings Commission, and the Committees to Investigate and Report on Methods of Snow Removal and Garbage Disposal. He represented the City of New York at the International

Road Congresses held in Paris, France, in 1908, in Brussels, Belgium, in 1910, and in London, England, in 1913, and at the International Congress of Cities in Ghent, Belgium, in 1913.

In 1910, he visited many European cities and reported on the character and control of their sub-surface structures and the use of pipe galleries for this purposes.

Mr. Lewis was one of a delegation of nine American engineers representing the National Engineering Societies, who visited France in December, 1918, at the invitation of the Société des Ingénieurs Civils de France and several of the Cabinet Ministers, to confer with committees of a French Congress under the chairmanship of M. Millerand, on problems of reconstruction following the World War.

After his retirement from the city service in 1920, Mr. Lewis had several months of well-earned rest in California and then became, in May, 1921, Director of the Physical Survey of the Regional Plan of New York and Its Environs under the auspices of the Sage Foundation. He began studies of physical and other conditions bearing on metropolitan development within an area of 5 528 sq. miles centering around New York City, and continued in this work (later, as Director of the Engineering Division) until his death.

He was the author of "The Planning of the Modern City", the second edition of which was published in 1923, and had contributed freely to the engineering press on matters relating to city planning, highways, and general civic improvement.

Nelson P. Lewis was one of the few engineers who have attained, in the grueling field of public service, both marked eminence in his profession and the admiration and affection of all those associated with him. He completed nearly two-score years of work in this service, filled with many unusual changes and difficulties, still carrying an unblemished reputation for integrity, loyalty, and professional efficiency. He had vision, appreciation, and interest in a high degree. He was genial, kindly, and helpful to all who sought his counsel or aid, an outstanding engineer, a warm friend and a delightful companion.

Mr. Lewis is survived by his wife, Minnie McLean Lewis, and one son, Harold M. Lewis, M. Am. Soc. C. E., who is Executive Engineer for the Regional Plan of New York and Its Environs.

Mr. Lewis was the first President of the Municipal Engineers of the City of New York and Past-President of the American Society for Municipal Improvements, of the American Road Builders' Association, of the Brooklyn Engineers' Club, and of the Brooklyn Municipal Club. In 1919-20, he was President of both the National Conference on City Planning and the American City Planning Institute. He was a Trustee of the Brooklyn Polytechnic Institute, a Trustee and Past Secretary of the Adirondack League Club, and a member of the Engineers' Club of New York City. He was also a very

active member of the Flatbush Dutch Reformed Church and served, at two different times, as an Elder on its Consistory.

Mr. Lewis was elected a Member of the American Society of Civil Engineers on October 2, 1895. He served as Director from 1904 to 1906 and as Vice-President in 1918 and 1919.

RICHARD DANA UPHAM, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 29, 1924.

Richard Dana Upham, the son of Dr. J. Baxter and Catherine Bell Upham, was born in the old Dana mansion, Manchester-by-the-Sea, Mass., on August 18, 1866. He was graduated from St. Mark's School and, later, from Harvard College in the Class of 1890.

Soon after his graduation, Mr. Upham was appointed to the position of General Superintendent of the International Pavement Company, New York, N. Y., which position he held until the fall of 1893. During this time he made a thorough study of the manufacture of asphalt for paving and other commercial uses and obtained the extensive knowledge of this material that enabled him to develop improvements in its manufacture and that eventually revolutionized the methods then in use.

Mr. Upham continued his investigations during the next three years, having accepted an appointment with the Barber Asphalt Company, in October, 1893. His work with this Company was interesting in its variety, including the erection of the plants at Long Island City, N. Y., on the Schuylkill River, Philadelphia, Pa., and the enlargement of the plant at Trinidad, British West Indies.

In 1896, he opened an office as Consulting Engineer, specializing, among other things, in water-proofing problems. Under Mr. Upham's supervision the Queen Lane and Roxborough Reservoir, in Philadelphia, was successfully repaired. He was also consulted on and directed the water-proofing of the Rapid Transit Tunnel in New York.

In 1912, he was one of the organizers of The Interocean Oil Company, of which he was Vice-President and General Manager, a position he held until his death.

On August 22, 1895, Mr. Upham was married to Elizabeth Rice, of New York, who, with a son and daughter, survives him. He died on September 29, 1924, and was interred in Mount Auburn Cemetery, Cambridge, Mass. Mr. Upham was a man of high ideals and absolute integrity, a loyal friend, and devoted to his family. He was a member of the University, Harvard, and Lawyers' Clubs and the New York and Seawanhaka Yacht Clubs. His favorite sport was sailing his schooner yacht *Sea Fox,* on which he lived during the summer. He always commanded her himself and on his last cruise, immedi-

* Memoir prepared by Chandler Davis, M. Am. Soc. C. E.

ately before his final illness, achieved the feat of bringing her safely through the hurricane of August 25, 1924, off the New England Coast.

Mr. Upham was elected an Affiliate of the American Society of Civil Engineers on October 6, 1896, and an Associate Member on April 3, 1901.

GEORGE LAWRENCE WATTERS, Assoc. M. Am. Soc. C. E.*

DIED MAY 18, 1924.

George Lawrence Watters was born at St. Clairesville, Ohio, on October 23, 1885. His paternal ancestors came to America in the early part of the Seventeenth Century and settled in Maryland and Virginia. His maternal ancestors who were French Huguenot and German, originally settled at Old Economy, west of Pittsburgh, Pa., and were members of the Harmony Society.

Mr. Watters' father, Leonidas Hamline Watters, spent the greater part of his life in educational work, having been assisted by his wife who was also a keen student and collaborated with him in his writings. Their children were surrounded by an atmosphere of culture and refinement, particular attention being given to the arts, literature, and science.

George Lawrence Watters received his early education in the public schools of Media, Pa., where his father held the position of Principal of the High School. After leaving the Media High School, he entered Swarthmore College at Swarthmore, Pa., and was graduated in the Class of 1906, with the degree of A. B. in Engineering.

Immediately after his graduation, Mr. Watters was employed as Assistant Engineer with the American Pipe and Construction Company of Philadelphia, Pa., on the design and construction of masonry and earth dams, reservoirs, stand-pipes, pipe lines, and filtration plants. As such, he served in many and varied capacities from Designing Draftsman to the actual execution and construction of the work in the field, as Resident Engineer.

In 1911, he resigned his position with the American Pipe and Construction Company and accepted that of Hydraulic Engineer with the Lehigh Valley Railroad Company, in which capacity he had charge of the design, construction, operation, and maintenance of the water supplies and fire protection of the Company, with headquarters at Bethlehem, Pa. During his employment with the Lehigh Valley Railroad Company he instituted many improvements in its water supply and installed a number of pumping and gravity supplies and water-softening plants.

In March, 1919, the position of Hydraulic Engineer having been abolished, Mr. Watters accepted that of Special Engineer of the International Petroleum Company, Limited, of Toronto, Ont., Canada, and made a three months' visit to the oil fields in Peru. The particular problem of the Company was to obtain a sufficient and adequate water supply for its operations, and as a

* Memoir prepared by J. E. Gibson, M. Am. Soc. C. E.

result of Mr. Watters' investigations he was sent to Peru in November, 1919, to install a complete water supply system, including the intake, high and low-lift pumping plants, a filtration plant, and a distribution system, with leveling and supply reservoirs and a stand-pipe. The chief difficulty in this work was to obtain sufficient water to meet the demands of the Oil Company with an available annual rainfall that seldom exceeded 2 in.; paradoxically as it may seem, during his engagement in Peru, he was called on to assist in the solution of a flood problem that occurred, due to an unprecedented rainfall or cloudburst.

Mr. Watters completed the water supply work and returned to the United States early in 1922. In June, 1922, after a vacation on the Pacific Coast, he made a third trip of three months to Peru and a fourth trip in January, 1923, for a period of six weeks. These latter trips were inspection, or consultation, trips with the Resident Engineers of the Oil Company in connection with water supply and flood-protection problems.

In May, 1923, Mr. Watters severed his connection with the International Petroleum Company, and accepted a position with the Western New York Water Company as Designing Engineer. He undertook and completed the design of the 16 000 000-gal. gravity and mechanical concrete filter plant a few weeks before his death.

In 1913, while employed with the Lehigh Valley Railroad Company, Mr. Watters was unfortunate enough to break his leg while playing baseball and was confined to the hospital for a period of approximately nine weeks. At that time he learned that he had developed kidney trouble, but as he was very optimistic, and reticent about the condition of his health, his friends and acquaintances were not informed as to his true condition. However, it was not until 1923 that his illness became really serious. He died in Bethlehem, Pa., on May 18, 1924.

To those who were associated with him in the employ of the American Pipe and Construction Company of Philadelphia, Mr. Watters endeared himself by his courtesy and consideration of others and his sterling qualities of character. He was painstaking and accurate in research, design, and construction, and was always ready to lend a helping hand in any task, whether in the drawing-room or in the field, and none ever called on him for assistance who did not receive the best he could give.

Mr. Watters was married on March 3, 1916, to Bertha Louise Hark, of Bethlehem. Two children were born to them, Alice, who died in July, 1917, and Louise.

He was a member of the American Water Works Association, the New England Water Works Association, the Engineers' Club of Philadelphia, and of the George W. Bartram Lodge No. 298 F. and A. M., at Media, Pa. He was also a member of the Presbyterian Church and took an active part in its affairs, serving as a Trustee for a number of years.

Mr. Watters was elected an Associate Member of the American Society of Civil Engineers on April 30, 1912.